A MODEL FOR STRUCTURAL EVALUATION OF CONCRETE RUNWAYS

A Thesis Submitted
in Partial Fulfillment of the Requirements
for the Degree of

MASTER OF TECHNOLOGY

by

Capt. Rajiv Bhola

to the

DEPARTMENT OF CIVIL ENGINEERING
INDIAN INSTITUTE OF TECHNOLOGY KANPUR
February, 1996

Dedicated

to

My parents and daughter

CERTIFICATE

It is certified that the work contained in the thesis entitled, **"A MODEL FOR STRUCTURAL EVALUATION OF CONCRETE RUNWAYS"**, has been carried out by Capt. Rajiv Bhola under my supervision and this work has not been submitted elsewhere for the award of a degree.

February, 1996

(Dr. Sudhir Misra)
Associate Professor
Department of Civil Engineering
Indian Institute of Technology Kanpur

ACKNOWLEDGEMENT

I express my sincere gratitude to my thesis supervisor, Dr. Sudhir Misra, Associate Professor in the Department of Civil Engineering, I.I.T Kanpur, for his valuable guidance and suggestions throughout the course of this study.

I am extremely grateful to the faculty members of I.I.T Kanpur, who taught me during the M. Tech. programme and specially to Dr. A.S.R. Sai and Dr. B.R. Marwah who have been a constant source of encouragement for me.

I wish to place on record my sincere thanks to the staff of SEMT Wing, College of Military Engineering, Pune, for providing me the required data.

Thanks are also due to my friends, Capt. K.C. Panchanathan, Mr. K.S.N. Gupta, Ms. Aparna, Ms. Varsha, Mr. S.B. Singh, Mr. K.S. Grover, Mr. B. Setia, and Mr.U.S. Dixit for their help, assistance and moral support during the course of my stay at the institute.

Finally, I am grateful to the Indian Army and the Corps of Engineers in particular for sponsoring me to the M. Tech. Programme.

CONTENTS

LIST OF FIGURES

LIST OF TABLES

SYMBOLS

A	Area of loaded area
a	Radius of loaded area
b	Radius of tire imprint
D	Flexural rigidity of concrete slab
d_z	Vertical deflection of a point in layered structure
d_o	Surface deflection
E_c	Elastic modulus of concrete
E_m	Elastic modulus of subgrade
E_o	Surface modulus of elasticity of transformed half space
FWD	Falling weight deflectometer
h	Thickness of concrete slab
h_e	Equivalent thickness of concrete slab in terms of subgrade
k	Modulus of subgrade reaction
l	Radius of relative stiffness
m	Types of aircraft
N_f	Allowable number of load repetitions
n	Number of actual load repetitions in an year
P	Total load
p	load intensity
r	Radial distance from center of loaded area
S_c	Modulus of rupture of concrete
x	Distance along the x-axis
Y	Number of years
Y_f	Fatigue life in years
Y_{rf}	Remaining fatigue life in years

y	Distance along the y-axis
z	Depth below the surface
δ	Displacement measured by a transducer
ε_z	Vertical strain
υ_c	Poisson's ratio of concrete
υ_m	Poisson's ratio of subgrade material
σ_c	Stress in concrete slab due to corner loading
σ_e	Stress in concrete slab due to edge loading
σ_i	Stress in concrete slab due to interior loading
σ_z	Vertical stress
ω	Deflection of concrete slab
ω_o	Deflection of concrete slab at a point immediately under the load
CI	Cracking Index
AACI	Average annual cracking index

ABSTRACT

A concrete runway pavement is designed to last for a specified period. During its service life, it is subjected to varying repetitive loads. Due to increasing loads of aircraft and their number of annual repetitions on the pavement, there is a possibility of premature structural distress. This requires periodic structural evaluation to asses the actual load carrying capacity of the pavement. Current methods of evaluation are principally based on empirical relationships and accumulated experience of pavement design.

In this thesis an evaluation model based on non-destructive testing by means of Falling Weight Deflectometer (FWD) has been presented. The approach adopted is to first estimate the insitu material properties (elastic moduli of concrete slab and subgrade) of the pavement structure using the FWD deflection data. Then using these moduli as indicators of insitu strength of the pavement materials, assessment of the structural capacity and remaining fatigue life of the concrete runway is made. The moduli are determined using the Method of Equivalent Thicknesses and Woinowsky-Kreiger's analysis of loaded plate over Winkler foundation. The residual fatigue life is estimated by carrying out analysis based on cumulative damage due to actual aircraft loads and their annual repetitions. Finally, the structural capacity of the pavement may be denoted based on the load repetitions expected in future and the fatigue life desired.

CHAPTER 1

INTRODUCTION

1.1 General

A concrete airport pavement consists of a relatively thin concrete slab with subbase or subgrade for support (Fig.1.1). For structural analysis of concrete pavements, various analytical models have been developed. In most of these models, concrete pavement is assumed as a plate resting on a Winkler foundation. The structural capacity of the pavement thus depends upon the strength of the concrete slab and the underlying subgrade. However, since the modulus of elasticity of concrete slab is much higher than that of the subgrade material, a major portion of its load carrying capacity is derived from the slab itself and largely depends upon its thickness and the flexural strength of the concrete.

Based on the flexural strength of the concrete used, the concrete runway pavement is designed for a particular design aircraft and its anticipated annual repetitions. During its service life the pavement is subjected to various aircraft loads which are repetitive in nature. Therefore it gradually undergoes structural deterioration due to fatigue cracking and climatic factors resulting in strength loss. Also there may be an increase in the load and number of repetitions of the aircrafts resulting in reduction of the pavement life. Therefore there is a need for

periodic structural evaluation of the pavement to :

1. establish its load carrying capacity for expected operations.
2. assess its ability to support significant changes from the anticipated volumes or type of air traffic.
3. determine the condition of existing pavement for use in the planning or design of improvements which may be required to upgrade the facility.

1.2 Methods of Structural Evaluation

Structural evaluation of a concrete pavement primarily involves determining the insitu strength of materials and then using these to find out its load carrying capacity and residual fatigue life. This includes determination of the flexural strength of concrete and the modulus of subgrade reaction. The thickness of the slab is usually available from the construction records. Where information is not available, it can be determined by boring in the pavement or other methods. The flexural strength of the concrete can be estimated by performing concrete compressive or splitting tests on the core samples taken from the airport pavement and then applying the available relationships between flexural strength and compressive strength or splitting strength. The modulus of subgrade reaction can be estimated by performing plate bearing tests on the subgrade. This process of estimating material properties of pavement structure results in holding up of air traffic for a considerable amount of time. However, in the present scenario where airports are busy and disruptions in air traffic cannot be afforded, other

non-destructive tests (NDT) for pavement evaluation have been developed.

These methods take into account the magnitude of stresses, strains and displacements arising out of a given load impressed on the pavement. But as the magnitude of stresses and strains induced in a pavement cannot be easily measured in the field, the measurement of pavement surface deflection is considered a more rational indicator of pavement performance.

The deflection of the concrete slab and its curvature under the load depend upon the strength, stiffness and thickness of the slab, subgrade moduli (or modulus of subgrade reaction) and the overall structural integrity of the pavement. Structural continuity conditions such as voids, cracks and joints also influence the deflection characteristics of the concrete pavement. Therefore, the measurements of the load deflection response of a rigid pavement is an adequate representation of pavement structural evaluation.

1.3 Types of Deflection Measuring Equipment

In general all deflection measuring equipment based on the types of load applied to the pavement viz, static or slow moving loads, steady-state vibration, and impulse loads, can be divided into three categories . In the first category, the Benkelman beam and California travelling deflectometer are the best known devices. The pavement deflections are measured by the rebound deflection of the equipment probe after a standard single-axle truck is slowly driven away. These devices suffer from the disadvantage that static or slowly moving loads do not

represent the transient or impulse loads actually imposed on pavements. Under the category of steady-state vibrations, Dynaflect and Road Rater are the best known devices. The deflections are generated by vibratory devices that impose a sinusoidal dynamic force over a static force and are measured by velocity sensors. In this case too the disadvantage is that the actual loads applied to the pavement are not in the form of steady-state vibrations [1]. Moreover these devices were developed for road pavements and had the limitation of measuring deflections under comparatively lesser load magnitude and for small pavement thickness, as compared to those encountered in airport runways.

For airport pavements, keeping in view the magnitude and the complex configuration of the aircraft landing gear, devices that deliver a transient force impulse to the pavement surface have been developed. All these devices come under the category of impulse load equipment, such as the various types of Falling Weight Deflectometers (FWD). By varying the amount of weight and the height of drop, different impulse forces can be generated. For ease of conducting tests, the device is mounted on a trailer. The normal operation is to move the trailer mounted device to the test location, lower the loading plate and transducers hydraulically to the pavement surface, complete the test sequence by dropping the weight at each height selected, lift the loading plate and sensors, and tow the device to the next site. The major advantage of the impulse loading device is its ability to accurately model an actual impact on the runway upon the landing of an aircraft in both magnitude and duration.

1.4 Evaluation of Pavement Using FWD

By using FWD equipment an impulse force is created by dropping a weight of 50 to 300 kg from a height of 20 to 500 mm [Fig.1.2]. By varying the drop height and weight, a peak force ranging from 7 KN to 125 KN can be generated. The load is transmitted to the pavement through a specially designed spring system and a hard rubber type loading plate, 300 mm in diameter. This arrangement provides a load pulse in the form of a half sine wave with a duration from 25 to 30 ms. The magnitude of load transferred to the pavement is measured by a load cell. Deflections are measured by transducers mounted on a bar that can be lowered automatically to the pavement surface with the loading plate. One of the transducer is located at the center of the plate, while the remaining can be placed at locations up to 2.25m from the center. Commercially available FWD is also usually equipped with a microcomputer which stores the deflections measured and the magnitude of the load transferred to the pavement.

The deflections measured by FWD loading are then used to estimate the moduli of concrete pavement components viz., modulus of elasticity of concrete, E_c and subgrade moduli, E_m. However, since the deflections depend upon a combination of E_c and E_m values, the same deflection response can be obtained for different values of moduli. This is discussed in greater detail in Chapter 2. Thus, in order to obtain unique values of E_m and E_c, the present study proposes a new method of estimating these parameters from the FWD deflection data.

After the insitu material properties of the concrete slab and the underlying subgrade have been estimated, the structural load carrying capacity and fatigue life of the pavement can be estimated based on the anticipated number of repetitions at different load levels. A methodology to denote the structural capacity of the pavement and asses its remaining fatigue life based on cumulative damage due to fatigue has been developed and discussed in Chapter 3.

1.5 Present Work

The aim of this thesis is to present a model for non-destructive structural evaluation of concrete runways. It seeks to develop :

(a) A model for estimation of E_m and E_c using the deflections measured by FWD loading.

(b) A methodology to assess the remaining fatigue life of the pavement using the values of insitu material properties obtained in (a) and the air traffic statistical data.

1.6 Layout of the Thesis

This thesis has been organized into four chapters.

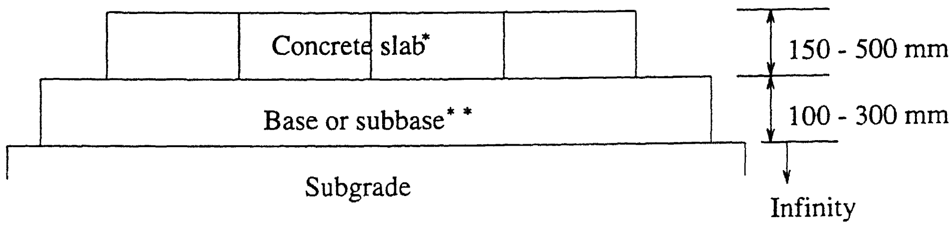
Chapter 1 presents a brief overview of the problem, the methodology and the scope of the present work.

Chapter 2 contains, in brief the current methods of estimation of elastic moduli and their disadvantages, development of models for estimation of subgrade modulus, E_m and modulus of

elasticity of concrete slab, E_c. It also presents verification of the proposed model.

Chapter 3 deals with the development of a methodology for assessing the remaining fatigue life of a pavement using the elastic moduli estimated and presents a proposal for denoting the structural capacity of the pavement, under evaluation, based on the anticipated repetitions and the fatigue life desired. To highlight the concept of fatigue in structural evaluation an illustrative example has also been presented.

Chapter 4 concludes the present work with recommendations and presents the future scope of study for civil engineers in this field.



* The runway can be considered to be made up of panels (6×8) m in size
** Sometimes ommitted

Fig. 1.1 Typical cross section of a concrete pavement used in runways

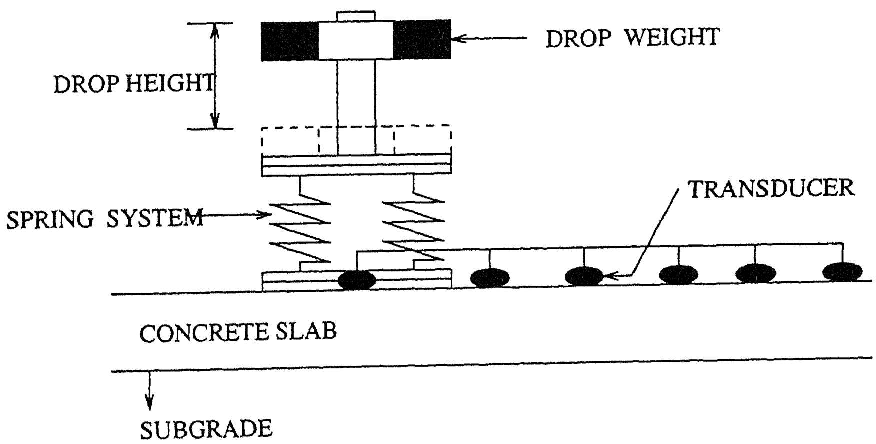


Fig. 1.2 Schematic representation of falling weight deflectometer (FWD)

CHAPTER 2

ESTIMATION OF SUBGRADE AND CONCRETE MODULI USING FWD DEFLECTION DATA

2.1 Background

As mentioned in Chapter 1, the FWD deflection data can be used to estimate the insitu material properties of the concrete slab and the subgrade viz., E_m and E_c. The methods are based on finite element methods (FEM). Earlier studies [2,3] indicate that the methods employ an iterative procedure in which the moduli are first seeded and then the actual values of elastic moduli of subgrade and concrete are obtained by matching the measured deflections with the computed deflections. Since the deflections depend upon a combination of E_m and E_c values, this method does not really yield unique values of moduli. This is illustrated by carrying out a parametric study, using the model as shown in Fig. 2.1a, to obtain the deflection response of a concrete pavement at different values of E_m and E_c. The pavement thickness and the load applied was maintained at 150 mm and 125 KN through out. The family of curves obtained are shown in Fig.2.1b It can be seen from Fig.2.1b that same deflection response can be obtained for different combinations of of E_m and E_c.

This led to further research. In China using FEM, numerous theoretical deflection data was generated for jointed

concrete pavement [3]. The theoretical deflections were used to develop predictive equations for estimation of subgrade elastic modulus. One of the predictive equation obtained was

$$\ln E_m = 5.39084 - 1.271741 \ln h - 1.422881 \ln \delta_1 \qquad (2.1)$$

where, E_m = Modulus of elasticity of subgrade in kg/cm^2

h = Thickness of concrete slab in cm

δ_1 = Deflection under the load for the interior FWD loading in cm

Using the above predictive equation, the test pavement deflections measured by FWD were used to evaluate the subgrade modulus. The value of E_m thus obtained was then seeded in the FEM based computer program to obtain the value of E_c. These estimated values of moduli, when compared with values determined by plate loading test on the surface of pavement and testing of core samples of concrete slab, showed that there was a variation of 300 to 400 percent.

To overcome the disadvantages in the present methods, a model based on analytical solutions has been developed to estimate E_m and E_c values.

2.2 Estimation of Subgrade Modulus, E_m

To estimate E_m the deflections due to the FWD loading requires modeling in terms of the subgrade material. This can be achieved by converting the concrete slab into an equivalent thickness in terms of the subgrade. The Boussinesq's equations

for computing surface deflections can then be used to compare the computed deflections with the measured deflections due to FWD loading and finally estimate the value of E_m.

2.2.1 Development of Proposed Model

The runway concrete pavement has been represented by a two layer structure. The concrete slab with finite thickness, lying on a semi-infinite space representing the subgrade. Both the layers are assumed to be homogeneous and isotropic. It is further assumed that the materials are elastic and linear.

The fundamental equations for determining stresses, strains and displacements in multi-layered system are based on Burmister theory and are often too complex. However, since the concrete runway is assumed to be only a 2-layer system, an alternative approach based on the transformation of the two layer elastic structure into an equivalent semi-infinite space by using the 'Method of Equivalent Thickness' (MET) has been developed. The Boussinesq's equations have been used for calculating the stress, strain and displacements [4].

The basic assumption in the Method of Equivalent Thickness is that the stress, strains and deflections below a layer for a given load will be unchanged as long as the flexural stiffness of the layer remains constant. The flexural stiffness is a function of the cube of the thickness of the layer, its modulus of elasticity and Poisson's ratio.

Now if the thickness of the concrete slab is h_1 then using the principle of equivalent thickness, this two-layer system can be transformed to a semi-infinite space, as shown in

Fig. 2.2, provided that the concrete layer is replaced by a thickness h_e of subgrade material having the properties of the semi-infinite space so that,

$$h_e = h_1 \times \left[\frac{E_c(1-\upsilon_m^2)}{E_m(1-\upsilon_c^2)} \right]^{1/3} \qquad (2.2)$$

where υ_m is the Poisson's ratio of the subgrade material and υ_c is the Poisson's ratio of the concrete.

(Concrete)	E_c, υ_c, h_1	$\Rightarrow$	h_e, E_m, υ_m	(Equivalent layer)
(Subgrade)	E_m, υ_m		E_m, υ_m	(Subgrade)

Fig. 2.2 Transformation of two-layer system

Once the two layer system has been transformed to a semi-infinite space, the stresses, strains and displacements can be calculated at any point using Boussinesq's equation [6]. For a system subjected to a point load, P the equations for vertical normal stress, σ_z, strain, ε_z, and deflection, d_z are given by :

$$\sigma_z = \frac{3P}{2\pi r^2} \cos^3\theta \tag{2.3}$$

$$\varepsilon_z = \frac{(1+\upsilon_m)P}{2\pi r E_m} (3\cos^3\theta - 2\upsilon_m \cos\theta) \tag{2.4}$$

$$d_z = \frac{(1+\upsilon_m)P}{2\pi r E_m} \left[2(1-\upsilon_m) + \cos^2\theta \right] \tag{2.5}$$

where r and θ are the polar coordinates of the point under consideration. For a uniformly distributed load, p on a circular area with radius, a, the variation with depth z along the center line of the load is determined by an integration of the Boussinesq's equations. The resulting equations are :

$$\sigma_z = P\left[1 - \frac{1}{\left[1+(\frac{a}{z})^2\right]^{2/3}}\right] \tag{2.6}$$

$$\varepsilon_z = \frac{(1+\upsilon_m)p}{E_m}\left[\frac{\frac{2}{a}}{\left[1+(\frac{z}{a})^2\right]^{3/2}} - (1-2\upsilon_m)\left[\frac{\frac{2}{a}}{\left[1+(\frac{z}{a})^2\right]^{1/2}} - 1\right]\right] \tag{2.7}$$

$$d_z = \frac{(1+\upsilon_m)pa}{E_m}\left[\frac{1}{\left[1+(\frac{z}{a})^2\right]^{1/2}} + (1-2\upsilon_m)\left[\left[1+(\frac{z}{a})^2\right]^{1/2} - \frac{z}{a}\right]\right] \tag{2.8}$$

Deflection of the surface can now be obtained by putting z=0 in eq.2.8. Deflection, d_o for the center and $d_o(r)$ for a distance r from the center of the loaded area can be given as :

$$d_o = \frac{2(1-\upsilon_m{}^2)pa}{E_o} \tag{2.9}$$

$$d_{o(r)} = \frac{(1-\upsilon_m{}^2)pa^2}{rE_{o(r)}} \tag{2.10}$$

where, E_o is the surface modulus of elasticity of the transformed system. Using eqs. 2.9 and 2.10, the surface deflections of the concrete pavement can thus be computed. It is also clear that if the surface deflections are known , the E_o of the transformed half space can be calculated using eqs. 2.9 and 2.10. It may now be recalled that the deflection data due to FWD loading gives precisely the d_o and $d_o(r)$. A representative data of deflections measured by FWD loading is presented in Table 2.1.

The stress zone when the pavement is loaded by FWD is shown in Fig. 2.3. The load is distributed through layers according to the broken lines. Clearly for a distance ($r > h_e$),

the deflection [$d_o(r)$] may be taken to be only depending upon E_m. This is clearly brought by calculating $E_o(r)$, i.e. the variation of E_o with the radial distance, r, as estimated by eq. 2.10. The variation is plotted in Fig. 2.4 and it can be seen that the value of E_o is maximum under the load and falls sharply as the distance from the load increases up to a certain limit and thereafter it stabilizes. The region where a more or less constant value of E_o is obtained r is larger than the equivalent thickness of the concrete slab. In this study the minimum value of E_o thus obtained has been taken to be equal to the modulus of subgrade, E_m.

It may, however be borne in mind that as the equivalent thickness of the concrete slab is not really known to begin with and there is thus a difficulty in choosing the location for the placement of transducers of the FWD. To overcome this a parametric study has been carried out to study the variation of equivalent thickness, h_e of the concrete slab with increase in the slab thickness, h at different values of E_m and E_c. The load imposed on the pavement has been assumed to be 125 KN as is the general case in FWD loading. It can be seen from Fig.2.5 that the value of equivalent thickness increases linearly with the slab thickness. However, the magnitude of increase depends on the combination of E_c and E_m values. An increase in the E_c value results in increased equivalent thickness whereas an increase in the E_m value leads to reduction in the value of equivalent thickness. In general, it is seen that for the slab whose thickness is 150 mm the equivalent thickness vary from 850 mm to 1600 mm for all values of E_c and E_m considered. And for 300 mm thick slab the variation of equivalent thickness is 1500 mm to

3300 mm. These values can be used as guidelines in placing the transducers of the FWD device suitably.

2.3 Estimation of Modulus of Elasticity, E_C of Concrete slab

In the previous section the deflections of the pavement due to FWD loading were modeled in terms of the subgrade material for estimation of E_m. To estimate E_c value of the concrete slab the FWD deflections require modeling in terms of the concrete parameters. For this purpose, as mentioned earlier, the concrete pavement is assumed to be an elastic plate resting on Winkler's foundation. The slab as well as the elastic foundation are assumed to be homogeneous and isotropic. It is further assumed that : (1) there is a complete contact between the slab and the subgrade, (2) the reaction of the subgrade on the slab is a vertical pressure, per unit area, and is constant called modulus of subgrade reaction, k.

Winkler's foundation concept assumes the foundation (subgrade) to behave as a dense liquid. Therefore the k value can be looked upon as a fictitious property and not the actual characteristic of the subgrade [5]. This is due to the fact that the subgrade is a solid in reality. However, due to simplicity in application, k value has been used most frequently for the design and evaluation of concrete pavements. Therefore it is considered useful if the k value can be related to the subgrade elastic modulus, E_m.

2.4 Relationship Between Modulus of Subgrade Reaction and Elastic modulus of Subgrade

One of the earliest empirical relationship between modulus of subgrade reaction, k and elastic modulus of subgrade, E_m was established by Hertz in 1922 [10] and is reproduced below

$$k = \frac{E_m}{2(1-\nu_c^2)} \qquad (2.11)$$

Huang [1] has stated that Vesic and Saxena in 1974, based on extensive experimental studies, indicated that the value of k depends on the relative flexibility of the slab with respect to the subgrade. They established following relationship for computing stresses underneath the concrete slab.

$$k = \left(\frac{E_m}{E_c}\right)^{1/3} \frac{E_m}{(1-\nu_c^2)h} \qquad (2.12a)$$

The above relationship when used for computing deflections underestimated the deflection values. Therefore for computing deflections Vesic and Saxena suggested that only 42 percent of the value obtained from eq. 2.12 be used. This relationship, as the literature suggests, has since been universally accepted [1] and has also been adopted for this thesis. In other words, the k has been taken as :

$$k = 0.42\left(\frac{E_m}{E_c}\right)^{1/3} \frac{E_m}{(1-\nu_c^2)h} \qquad (2.12b)$$

2.5 Studies on Analytical Solutions

Numerous theoretical analysis of loaded plates over elastic foundation exit in literature e.g. by Navier, Westergaard, Woinowsky - Kreiger, Hertz, Huang etc. However, it is felt that the Westergaard's analysis of stress concentrations in plate on elastic foundation loaded over small areas [7,8,9] and Woinowsky-Kreiger's analysis of a plate resting on elastic foundation loaded at equidistant points [10] are the two analysis which are directly relevant to studies concerning runways and are therefore explained in greater detail in the subsequent paragraphs. Both methods are applicable for analysis of large slabs. In this context it may be noted that a runway is made up of several panels jointed along the edges (Fig. 2.6). Considering the size of the panels (generally 6m × 6m to 8m × 8m) in comparison with the FWD loaded area, each panel has been assumed to be a large concrete slab.

2.5.1 Westergaard's Analysis

The Westergaard's analysis for computing stresses and its application to concrete pavements of airfield is well accepted. It is also the basis for evolution of pavement design curves, its acceptance in computing deflections has little mention in the literature reviewed. The analysis, for computing stresses and deflections, considers three different cases of load application viz. load applied at the corner, in the interior of a

slab at a considerable distance from any edge, and near the edge far from any corner (Fig.2.6). For estimation of representative E_c value of a concrete slab it is sufficient to consider the deflections due to interior loading case. The FWD loading is also applied at the interior of the pavement panel. The deflection equation presented by Westergaard in case of the load applied in the interior of the pavement panel is reproduced below :

$$\omega = \frac{P}{8kl^2} - \frac{P}{8\pi kl^4}\left[\frac{1}{A}\int r^2 dA \log\frac{1}{r} + \frac{1.1159}{A}\int r^2 dA + (K+0.1159)(x^2+y^2) + \frac{1}{2} S(x^2+y^2)\right] \tag{2.13}$$

where, P = Total load distributed uniformly over any area A having both axes of x and y as axes of symmetry.

l = Radius of relative stiffness, defined as

$$l^4 = \frac{E_c h^3}{12(1-\nu_c^2)k} \tag{2.14}$$

K and S are area coefficients defined as

$$K = -\frac{1}{A}\int dA \log 1/r \tag{2.15}$$

$$S = -\frac{1}{A}\int dA \cos 2\theta \tag{2.16}$$

If the loaded area is considered to be circular with radius 'a' and load intensity p (as is the case in FWD). Then evaluating coefficients K and S and using the fact that y=0 along the

X-axis, the eq.2.13 is reduced to the form

$$\omega = \frac{p\pi a^2}{4}\left[\frac{3(1-\nu_c^2)}{E_c h^3 k}\right]^{1/2} - \frac{0.275(1-\nu_c^2)p\pi a^2}{E_c h^3}\left[\frac{a^2}{2} + x^2\right]\log_{10}\frac{E_c h^3}{ka^4} - \frac{0.239(1-\nu_c^2)p\pi a^2}{E_c h^3}\left(\frac{6a^2}{8}\right) \qquad (2.17)$$

where, x is the distance from the center of the loaded area.

Using eq. 2.17 deflections were computed under the load and also at a distance of 200 mm, 300 mm, 900 mm, 1200 mm and 1500 mm from the center of the loaded area for 20 slabs of an airport and these computed values compared with the measured FWD deflections. The results are presented in Appendix A and reproduced for few slabs in Table 2.2 and Fig.2.8.

It is observed from Fig.2.8 that the Westergaard's formula for computing deflection gives value which is comparable with the FWD measured deflection only in the loaded area and in the near vicinity of the loaded area. For distances in excess of 400 mm from the center, the deviation of the computed values from the measured values is significant and in fact too much. Considering the deflections under the load only, the average difference in computed deflections when compared with the measured deflections due to FWD loading is 9.71 percent.

2.5.2 Woinowsky-Kreiger's Analysis

Woinowsky-Kreiger's analysis [10] is based on a large slab resting on Winkler's foundation and impressed by an infinite

number of point loads spaced at equal intervals along the center line of the slab (Fig.2.9). A displacement solution was assumed in the form of a trigonometric series, and the final result for theoretical deflections were established for a single load by letting the distance between the loads go to infinity. The deflection underneath the slab can be represented as given below

$$\omega = \omega_o + \frac{P\lambda^2}{ak}\sum_{m=2,4..}^{\infty} \frac{(-1)^{m/2}}{(\lambda^4+\mu_m^4)^{0.5}} \operatorname{Cos}\frac{m\pi x}{a}\, e^{-\beta_m y}(\gamma_m \operatorname{Cos}\gamma_m y + \beta_m \operatorname{Sin}\gamma_m y) \tag{2.18}$$

where, $\omega_o = \dfrac{P\lambda}{2\sqrt{2}ak} e^{-\lambda y/\sqrt{2}}\left[\operatorname{Cos}\dfrac{\lambda y}{\sqrt{2}} + \operatorname{Sin}\dfrac{\lambda y}{\sqrt{2}}\right]$; (2.19)

$\lambda^4 = \dfrac{k}{D}$; $\mu_m = \dfrac{m\pi}{a}$; $2\beta_m^2 = (\mu_m^4+\lambda^4)^{0.5} + \mu_m^2$;

$2\gamma_m^2 = (\mu_m^4+\lambda^4)^{0.5} - \mu_m^2$; and

$D = \dfrac{E_c h^3}{12(1-\nu_c^2)}$, flexural rigidity of the concrete slab.

To compute the deflections along the x-axis, substituting y=0 and eq. 2.19 in eq. 2.18 gives

$$\omega = \frac{P\lambda}{2\sqrt{2}ak} + \frac{P\lambda^2}{ak}\sum_{m=2,4..}^{\infty} \frac{\gamma_m(-1)^{m/2}}{(\lambda^4+\mu_m^4)^{0.5}} \operatorname{Cos}\frac{m\pi x}{a} \tag{2.20}$$

In eq. 2.20 the value of 'a' to be substituted will

depend upon the size of the pavement panels. Normally the runway panels are 6m × 6m to 8m × 8m in dimensions therefore 'a' has been taken as 16 in the present work. Also it was found that the series converged well by taking 100 terms. Deflections were computed under the load by putting $x=\frac{a}{2}$ and at a distance of 200 mm, 300 mm, 900 mm, 1200 mm, and 1500 mm, by substituting appropriate values of x. These computed values were then compared with the measured deflections due to FWD loading. The results are presented in Appendix A and reproduced in Table 2.3 and Fig.2.10 for few slabs.

Comparing the computed deflections with the FWD measured deflections, it is observed that the Woinowsky - Kreiger's analysis yields theoretical deflections more closer to the measured ones and unlike Westergaard's solution the deflections are comparable over a larger range of distance away from the load. This can be seen in Fig. 2.10. Also the average difference in computing the deflection of the point immediately under the load in this case is about 5 percent as compared to about 10 percent in case of the Westergaard's analysis.

2.6 Development of Model for Estimation of E_c

From the foregoing it is clear that Woinowsky-Kreiger's analysis is better suited for evolving a method so that FWD measured deflections could be easily used in estimating the modulus of elasticity of the concrete slab. However, it may be noted that assuming the plate to be very large neglects the discontinuities resulting from transverse and longitudinal joints. This could be one of the reasons for the observed

difference in the deflection values measured by FWD loading and that computed using eq. 2.20. Also it should be noted that though the deflections are measured at the surface the computed deflections are the displacements of a point at the bottom of the concrete slab.

It can also be seen from Fig. 2.10 and Table 2.3 that instead of considering the entire deflection data, it is sufficient to compare the deflections at a point immediately under the load. This would also simplify the calculations. Since some difference in the computed and measured deflections exit, a parametric study has been carried out to study the deflection response of a pavement at a point immediately under the load using eq. 2.20. Further different values of E_m and E_c for varying slab thicknesses have been considered. It has been found (Fig. 2.11) that at all slab thicknesses and E_m values considered, a difference of about 5 percent in computed deflection result in about 15 percent of variation in estimation of E_c values or alternatively one percent of difference in computed deflections would lead to approximately three percent of error in estimation of E_c value.

To develop a reliable method, the surface deflections measured by FWD loading immediately under the load were compared with the computed deflections using eq. 2.20 for 20 slabs. A coefficient was then determined for each slab case to match the measured deflection with the computed deflection. The resulting average multiplying coefficient to be applied to the measured deflection comes out to be 1.05. The statistical results are tabulated in table 2.4. The coefficient determined has a standard deviation of 0.007. It is also found that this coefficient yields

an average difference of less than one percent between the computed and measured deflections may therefore be adopted for further computation.

From eq. 2.20 it can be seen that once the measured deflection under the load is known, E_c can be evaluated. But since it is contained in a series, analytical evaluation of E_c is not so easy. Therefore an iterative procedure using the optimization technique of interval halving has been used for determination of E_c. This procedure requires upper and lower limit of E_c values to be fixed beforehand. These have been taken as 85 GPa and 27 GPa respectively corresponding to the flexural strength of 3 MPa and 10 MPa. Also, k is required to be substituted in terms of E_c and E_m using 2.12b as mentioned earlier. The E_m value is the one obtained by using the method of equivalent thickness.

2.7 Verification of The Model

The overall model thus developed for estimation of E_m and E_c using the FWD deflection data can be represented as shown in Fig. 2.12. To obtain the deflection data, field measurements using FWD were conducted on an in service concrete pavement of an airfield. The pavement panels are 8m $\times$ 8m in size.The tests were conducted in May 95 when the ambient temperature was between 32 to 35 degrees centigrade. All the measurements were completed in one day. The thickness of the concrete slab as obtained from the construction records is 152 mm.

The deflection data obtained for 20 different concrete slabs has been used to estimate the E_m and E_c values with the help of the overall model developed in this work and the results compared with the results of the microcomputer based software (FWD package) available with the commercial FWD device. The results are presented in Table 2.5.

2.8 Discussion on results

It can be seen from Table 2.5 that the estimation of E_m and E_c values using the model developed check very closely with those obtained using the commercially available FWD package. The average difference in estimation of E_m and E_c is 7 percent with the standard deviation of 0.04 and 2 percent with the standard deviation of 0.02 respectively. The new method developed has the advantage that it estimates E_m independently thus resulting in unique values of the elastic moduli. Further, the computer program developed with this thesis can be used independently to analyze the deflection data obtained after the FWD testing on any personal computer. This would enable the FWD equipment based microcomputer to be available for another testing at the same time. Also as the method is based on analytical solutions it requires comparatively less time and space for computation than an FEM based program.Though the results check well with the FWD package results it is recommended that these be independently checked with the experimental results for a wider range of slab thicknesses.

2.9 Arriving at Representative Elastic Moduli for the Pavement

The model proposed above estimates the elastic moduli (E_m and E_c) for each slab. However, to determine the structural capacity of the pavement considering it as one structural unit, there is a need to arrive at the elastic moduli values which may be representative of the pavement as a whole. It can be seen from Table 2.5 that for the 20 slabs of the airport pavement considered, the estimated E_m ranges from 75 MPa to 187 MPa with the mean of 122 MPa and the standard deviation of 28.35. Similarly estimated E_c ranges from about 31.4 GPa to 83.7 GPa with the mean of 46.15 GPa and the standard deviation of 13.37. This clearly brings out that merely taking the mean values may not represent the structural integrity of the pavement in the true sense. Therefore it suggested that a study based on adequate number of slabs and for a wider range of slab thicknesses be carried out to arrive at an appropriate statistical analysis method for this purpose.

3.0 Summary

Accurate estimation of elastic moduli of pavement structure is an important factor in pavement evaluation because these are the indicators of the insitu strength of pavement materials. This chapter presents the models developed to estimate them by analyzing the deflections measured due to FWD loading, a non-destructive means of testing. Important points indicated in this chapter are :

1. To estimate the elastic modulus of subgrade E_m, the pavement structure may be assumed to be a two layer structure comprising of concrete layer and the subgrade. By using the method of equivalent thickness the concrete layer is converted into the subgrade material. This transformation provides for the direct application of Boussinesq's deflection equation on the transformed half-space.
2. The surface modulus of elasticity E_o can then be computed using the measured deflections of the pavement, under evaluation, due to FWD loading. It is seen that the value of E_o is minimum at a point where the radial distance from the center of the loaded area is just greater than the equivalent thickness. As, at this point the effect on the deflection is only due to the subgrade, the value of E_o is equal to the value of E_m. This is estimated E_m.
3. Through parametric studies it is also found that the equivalent thickness of the concrete linearly increases with the slab thickness. The magnitude of the increase depends on the relative stiffness of the concrete slab with subgrade.
4. To estimate the modulus of elasticity of concrete slab the center deflection measured due to FWD loading has been used. The already estimated value of E_m is first substituted in the Woinowsky-Krieger's deflection equation for computing deflection. An iterative procedure is then adopted to find out the value of E_m by matching the computed and measured deflections.

5. The study carried out also reveals that one percent of error in measuring the actual deflection leads to an error of three percent in estimation of E_c by using the model developed.

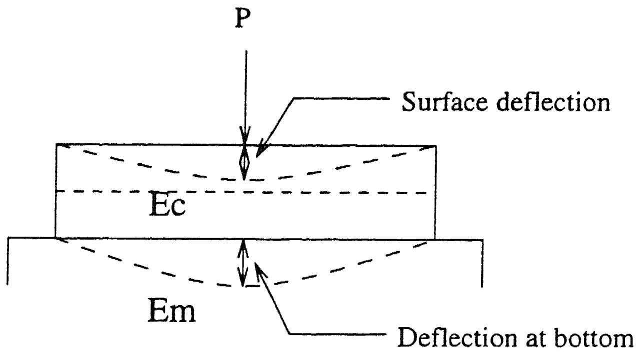


Fig. 2.1a Deflection of concrete slab under load application

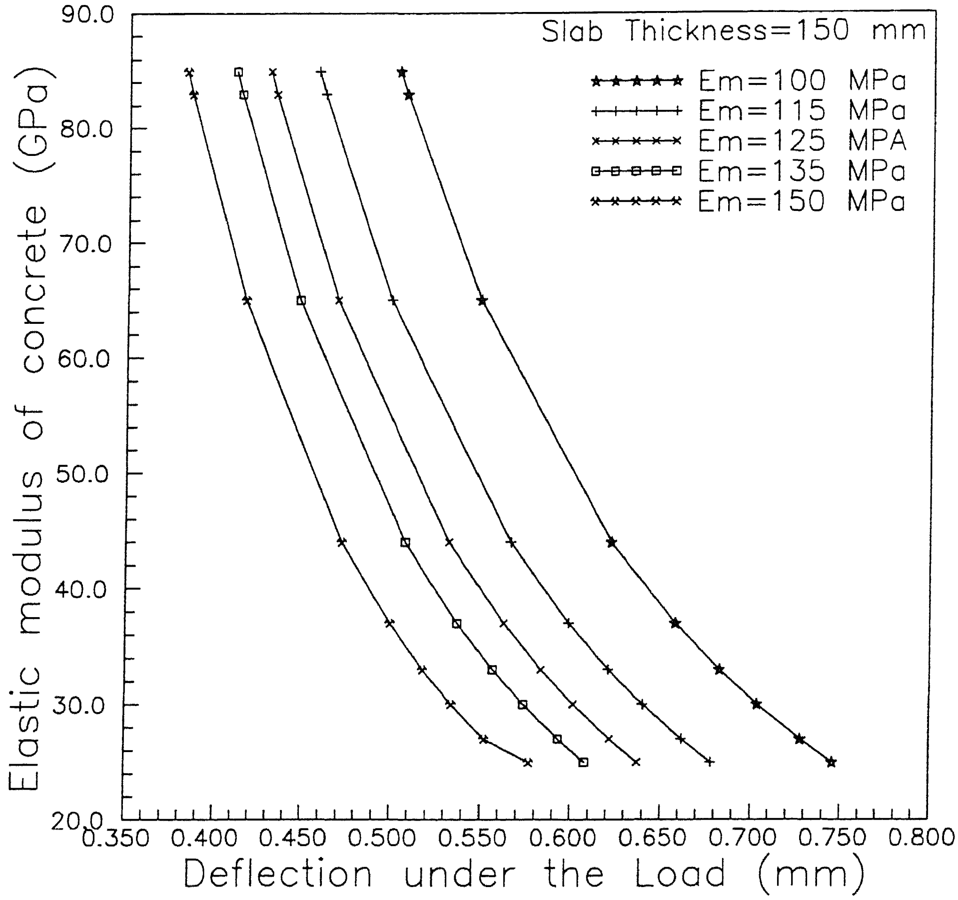
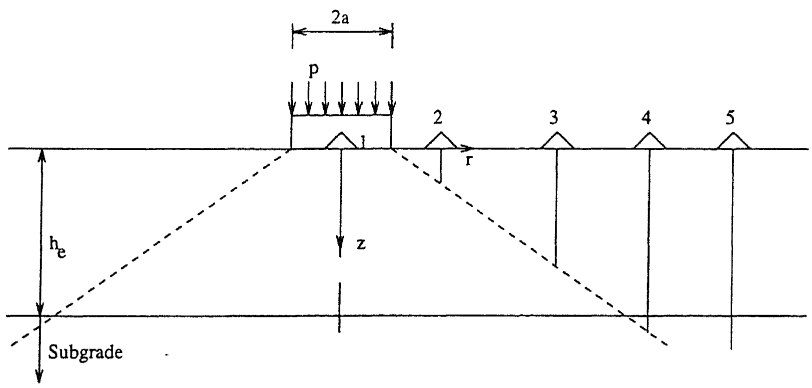
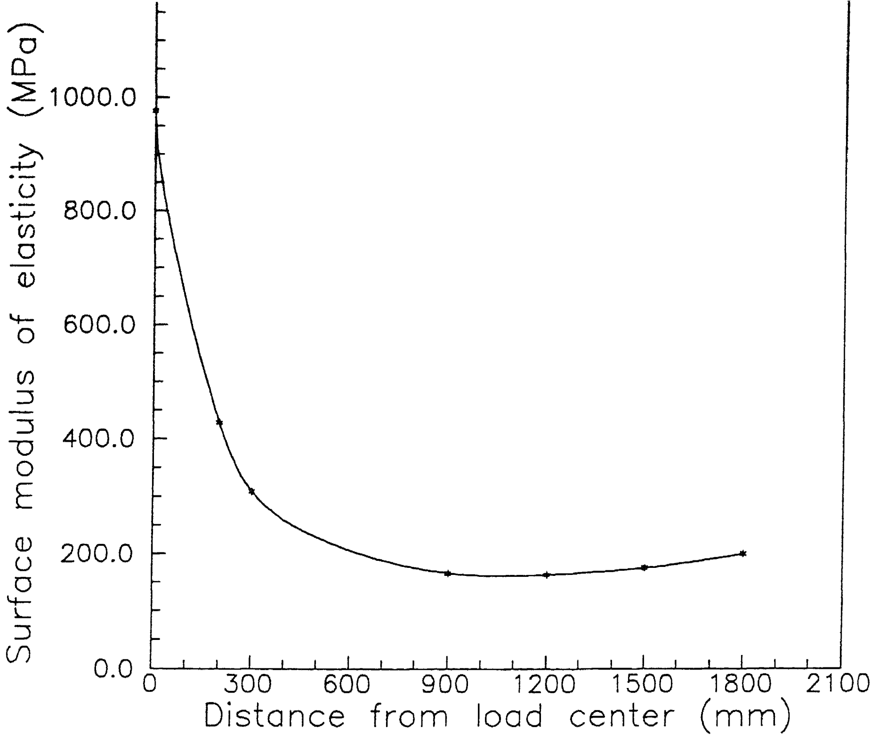


Fig. 2.1b Effect of elastic moduli, E_m and E_c, on the deflection of concrete pavement

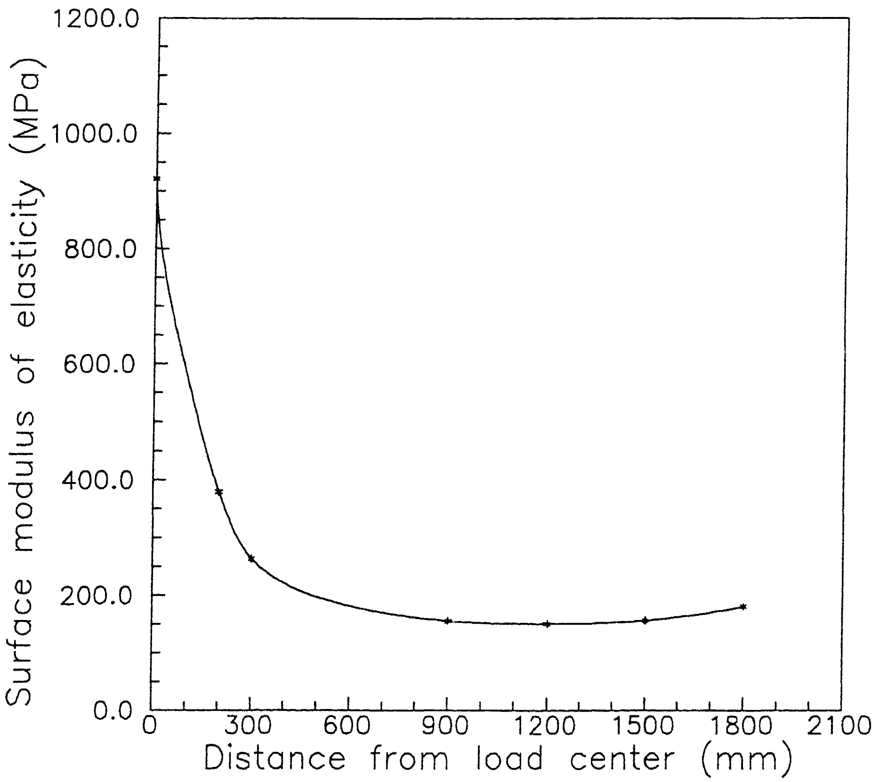


△1 - △5 : Transducers for deflection measurement

Fig. 2.3 Model of stress zone within pavement when loaded with FWD



(a)



(b)

Fig. 2.4 Variation of surface modulus of elasticity, E_o, with radial distance, r from the center of the loaded area

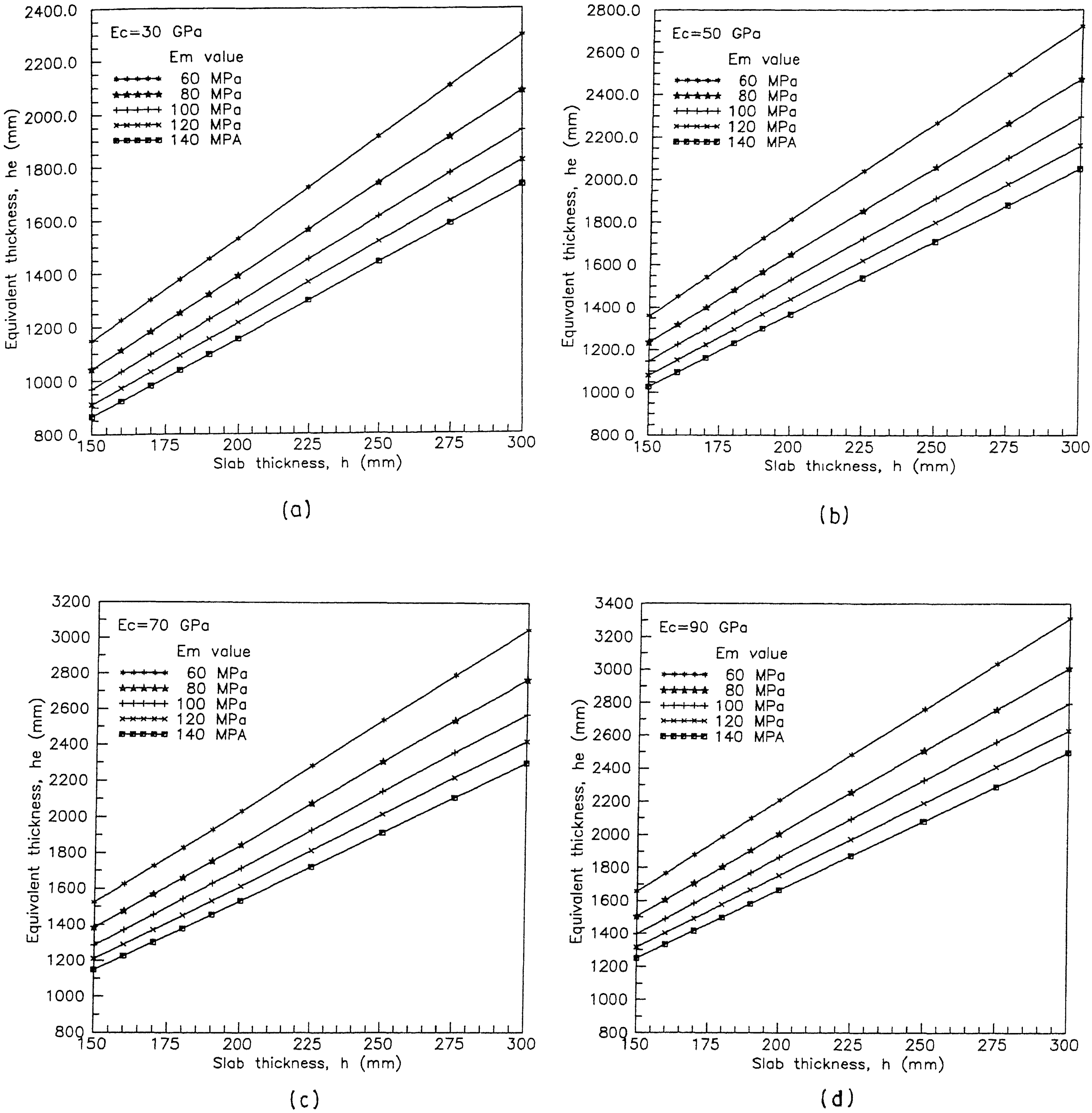


Fig. 2.5 Effect of slab thickness and elastic moduli on the equivalent thickness of the concrete slab in terms of the subgrade

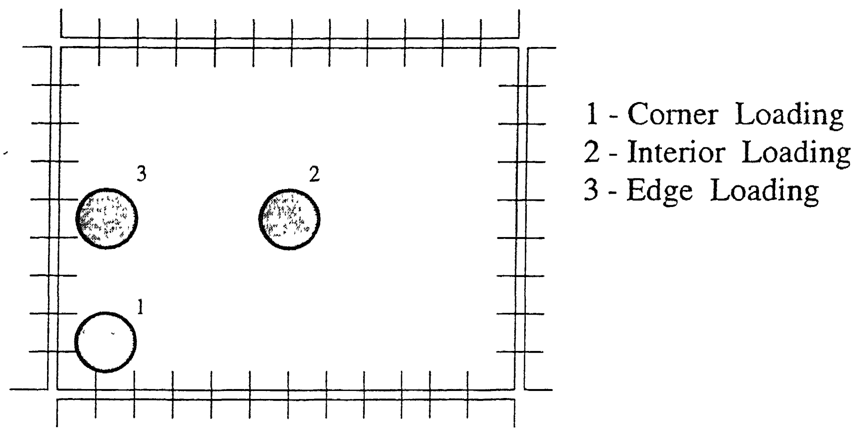


Fig. 2.6 Possible locations of load application on a panel.
[The Pavement consists of several panels jointed along the edges using dowel bars as shown]

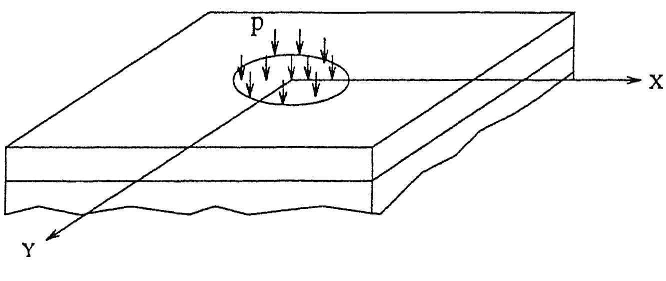


Fig. 2. 7 Representation of Westergaard's model showing uniform circular loading on an elastic plate over elastic foundation

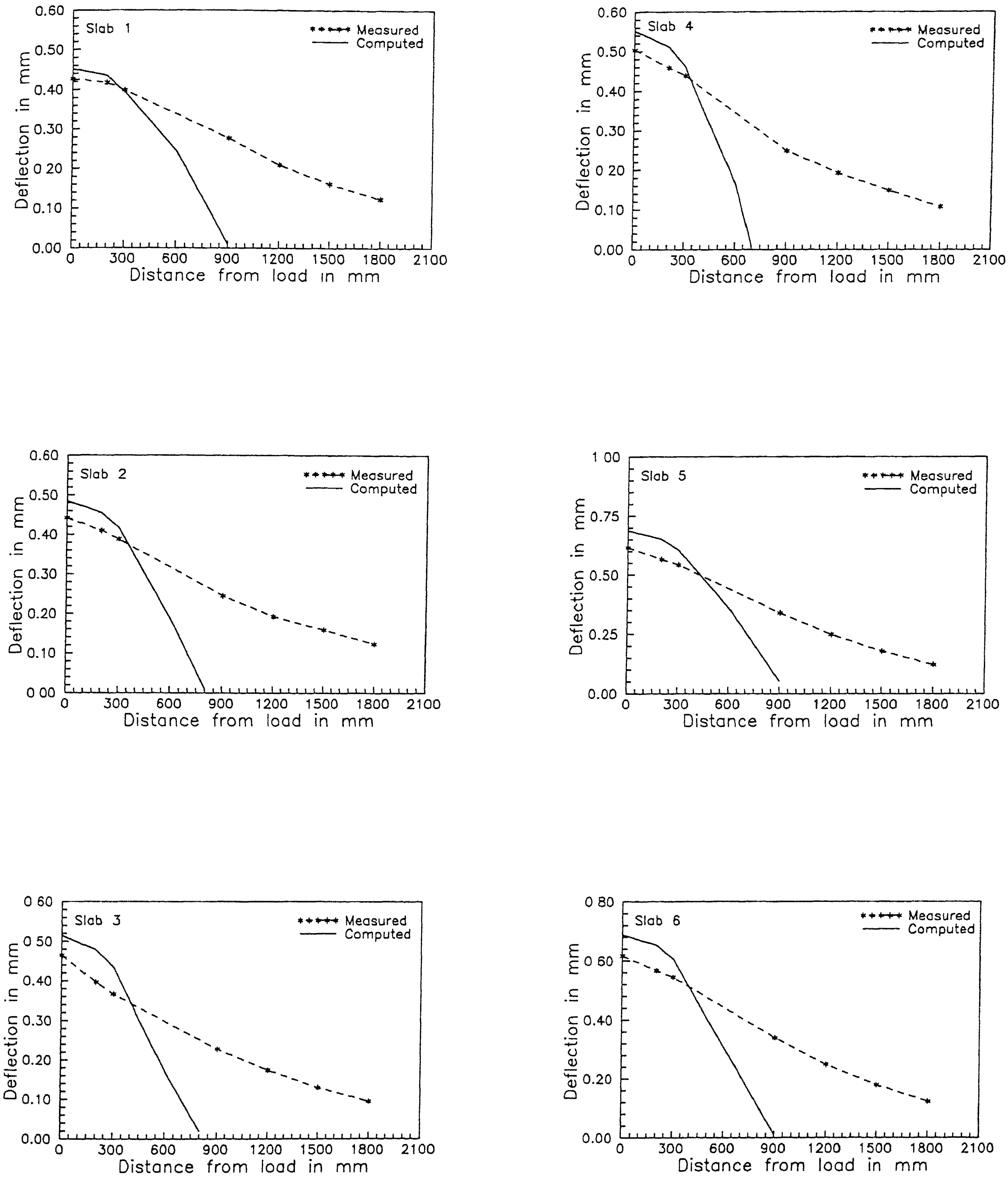


Fig. 2.8 Comparison of the estimated deflection using Westergard's model with the actually obtained values for different slabs

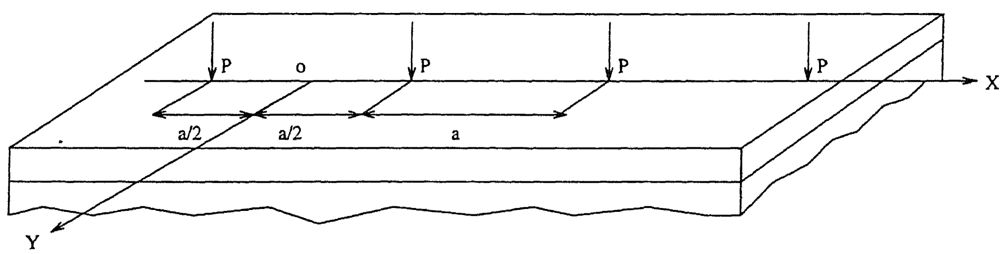


Fig. 2.9 Representation of Woinowsky - Kreiger's model showing applicatioof concentrated loads at equal intervals on elastic plate over elastic foundation

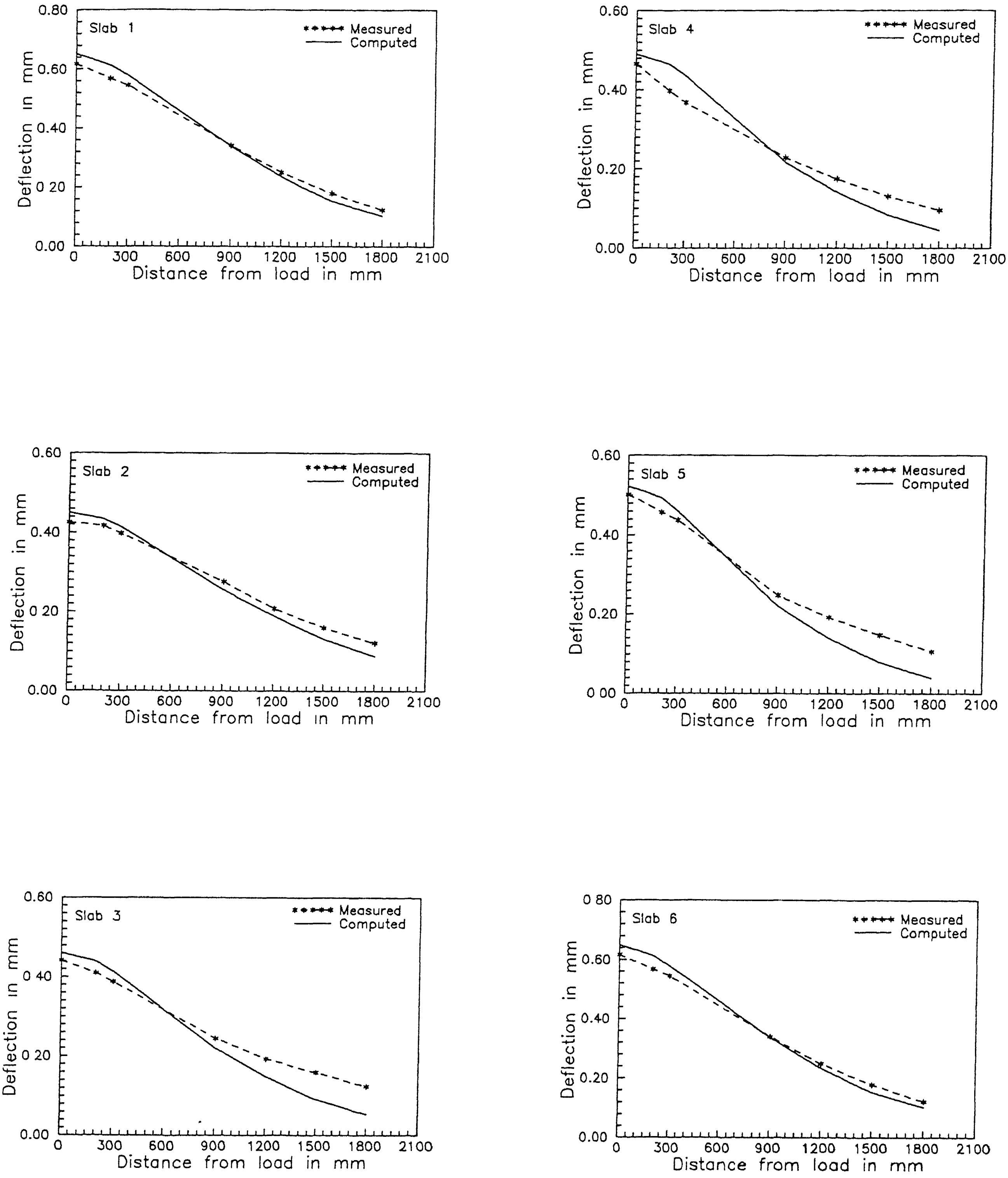


Fig. 2.10 Comparison of the estimated deflection using Woinowsky- Kreiger's model with the actually obtained values for different slabs

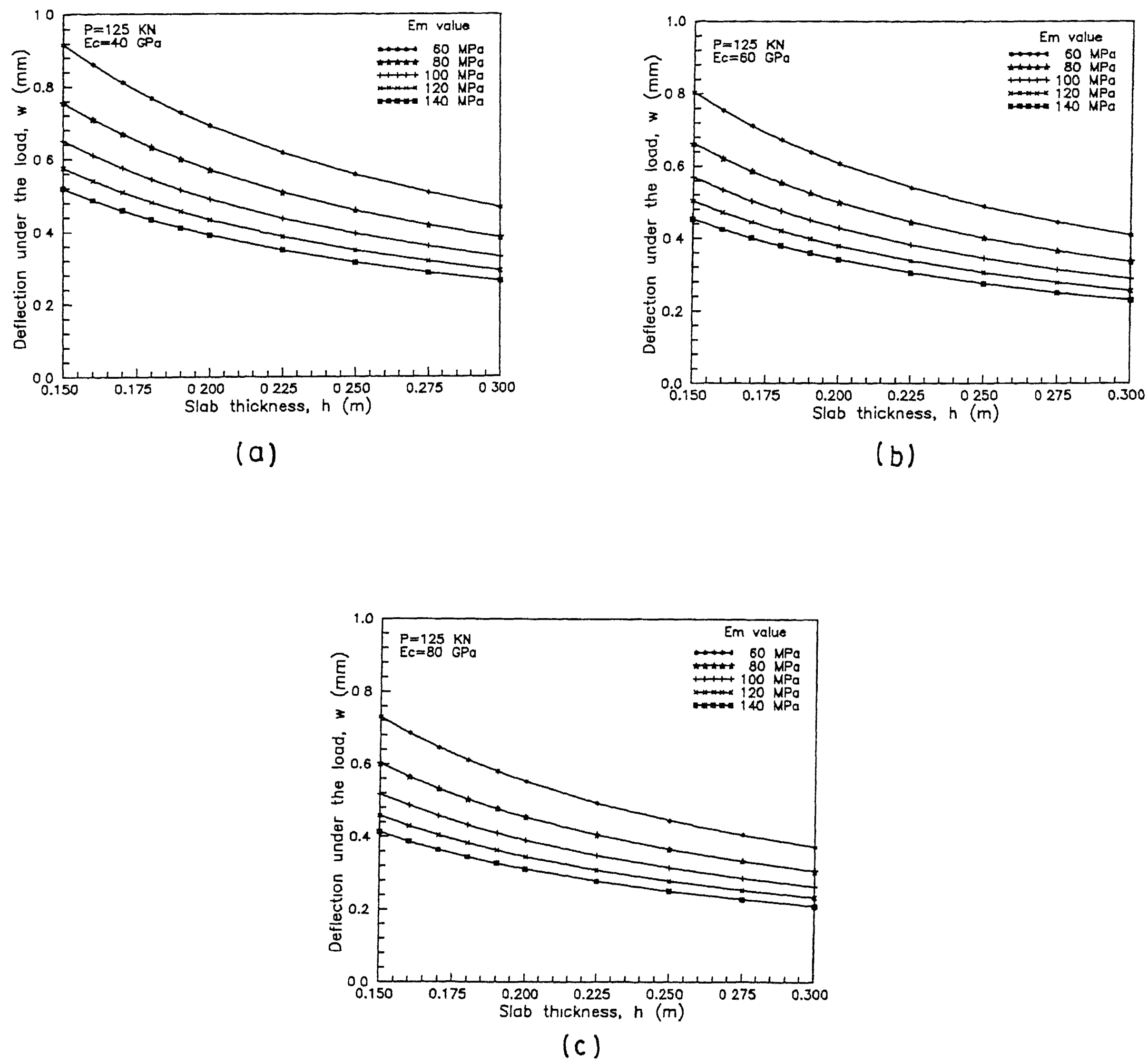


Fig. 2.11 Deflection of concrete pavement due to FWD loading for different slab thicknesses and elastic moduli

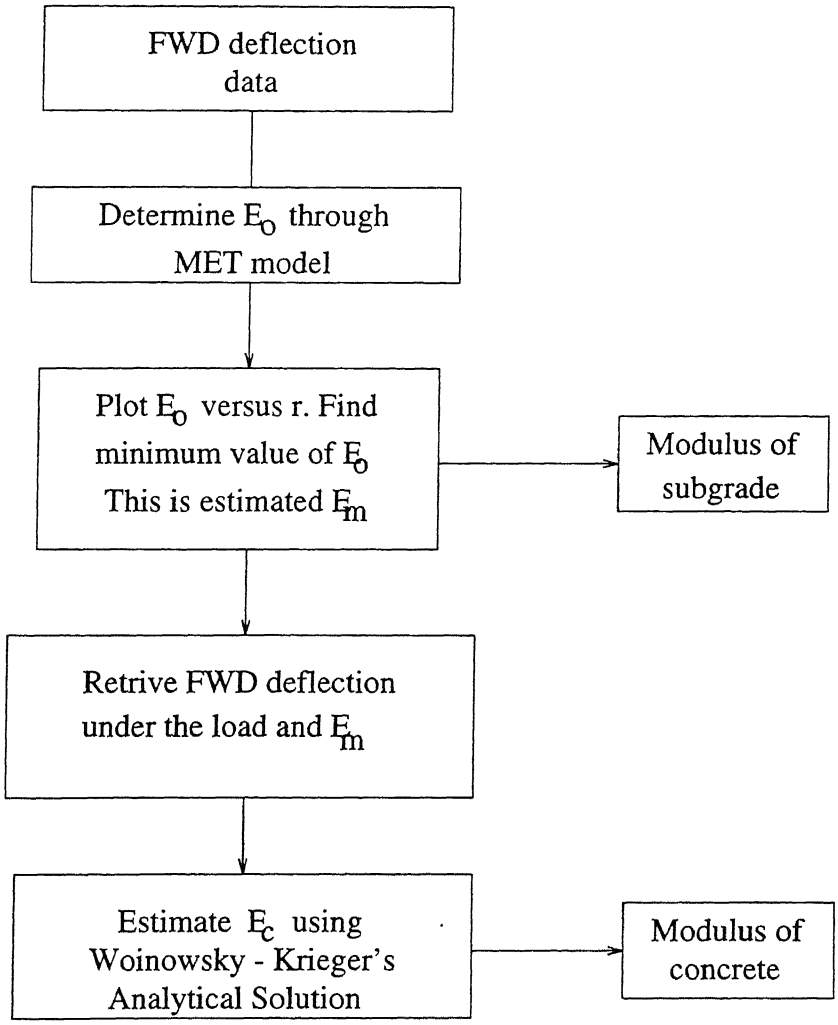


Fig. 2.12 Estimation of elastic moduli of subgrade and concrete

Table 2.1 Representative FWD deflection data

Stn: 3.10 Load (KPa): 1827 Tmp (celsius): 35

Deflections (μm):

Df1	Df2	Df3	Df4	Df5	Df6	Df6
363	359	297	204	166	141	128

Note: Df1, Df2 ... are the transducers and their distances from the center are preset at 0, 200, 300, 900, 1200, 1500 and 1800 mm.

Table 2.2 Comparison of deflections (Westergaard Vs FWD)

Slab number	Material properties	Parameter	Distance from center of the load(mm)					
			0	200	300	900	1200	1500
1	Ec=65658	measured	0.426	0.417	0.398	0.276	0.208	0.160
	k =63.93	computed	0.451	0.434	0.394	-0.004	-0.352	-0.801
2	Ec=44661	measured	0.442	0.410	0.388	0.245	0.193	0.159
	k =87.18	computed	0.486	0.455	0.417	-0.129	-0.608	-1.223
3	Ec=37509	measured	0.465	0.397	0.367	0.228	0.174	0.130
	k =90.76	computed	0.515	0.479	0.435	-0.199	-0.755	-1.469
4	Ec=33172	measured	0.503	0.458	0.438	0.248	0.192	0.148
	k =94.56	computed	0.551	0.511	0.461	-0.258	-0.887	-1.696
5	Ec=38931	measured	0.617	0.567	0.544	0.340	0.249	0.179
	k =49.38	computed	0.689	0.653	0.607	-0.048	-0.621	-1.368

Notes : 1. Ec is the modulus of elasticity of concrete (MPa)
2. k is the modulus of subgrade reaction (MN/m^3)
3. Deflections (mm)

Table 2.3 Comparison of deflections (Woinowsky-krieger Vs FWD)

Slab Number	Material properties	Parameter	Distance from center of the load(mm)					
			0	200	300	900	1200	1500
1	Ec=65658	measured	0.426	0.417	0.398	0.276	0.208	0.160
	k =63.93	computed	0.444	0.421	0.399	0.245	0.176	0.120
2	Ec=44661	measured	0.442	0.410	0.388	0.245	0.193	0.159
	k =87.18	computed	0.466	0.434	0.405	0.214	0.138	0.083
3	Ec=37509	measured	0.465	0.397	0.367	0.228	0.174	0.130
	k =90.76	computed	0.492	0.455	0.422	0.211	0.131	0.074
4	Ec=33172	measured	0.503	0.458	0.438	0.248	0.192	0.148
	k =94.56	computed	0.523	0.481	0.444	0.212	0.127	0.069
5	Ec=38931	measured	0.617	0.567	0.544	0.340	0.249	0.179
	k =49.38	computed	0.651	0.613	0.578	0.337	0.234	0.153

Notes : 1. Ec is the modulus of elasticity of concrete (MPa)
2. k is the modulus of subgrade reaction (MN/m^3)
3. Deflections (mm)

Table 2.4 Comparison of deflections under the load and determination of multiplying coefficient

Slab number	Measured (mm)	Computed (mm)	Multiplying coefficient
1	0.401	0.423	1.054
2	0.426	0.444	1.042
3	0.442	0.466	1.054
4	0.465	0.492	1.058
5	0.503	0.523	1.039
6	0.618	0.652	1.055
7	0.616	0.654	1.062
8	0.617	0.651	1.055
9	0.498	0.530	1.065
10	0.529	0.554	1.047
11	0.804	0.850	1.057
12	0.729	0.769	1.055
13	0.732	0.760	1.038
14	0.444	0.467	1.052
15	0.504	0.534	1.056
16	0.497	0.524	1.054
17	0.419	0.441	1.053
18	0.541	0.573	1.059
19	0.507	0.534	1.054
20	0.584	0.621	1.063

Mean = 1.05355
Standard deviation = 0.00725

Table 2.5 Comparison of E_m and E_c values

Slab number	Estimated* E_{m1}	Estimated* E_{c1}	Predicted by proposed method E_{m2}	Predicted by proposed method E_{c2}	$\frac{E_{m1}}{E_{m2}}$	$\frac{E_{c1}}{E_{c2}}$
1	126	83.423	131	83.665	0.96	0.99
2	130	65.658	136	63.505	0.95	1.03
3	149	44.661	154	44.723	0.97	0.99
4	147	37.509	153	37.956	0.96	0.98
5	147	33.172	150	31.755	0.98	1.04
6	102	33.113	106	33.073	0.96	1.01
7	97	36.681	105	37.498	0.92	0.98
8	94	38.931	110	38.958	0.85	0.99
9	110	51.183	127	52.453	0.86	0.98
10	113	46.829	122	45.914	0.93	1.02
11	69	32.515	78	32.699	0.89	0.99
12	68	45.223	75	45.279	0.92	0.99
13	77	35.547	93	33.898	0.83	1.05
14	126	59.216	135	58.616	0.93	1.01
15	99	61.709	110	62.694	0.90	0.98
16	146	33.857	154	33.813	0.95	1.01
17	173	40.504	187	40.330	0.93	1.01
18	98	54.093	102	54.692	0.96	0.98
19	129	40.564	132	40.454	0.98	1.01
20	89	49.760	92	51.016	0.96	0.98
				Mean	0.94	0.98
				Standard deviation	0.04	0.02

Notes: 1. * by the commercial software available with the FWD equipment.
2. Values of elastic moduli of subgrade (E_{m1}, E_{m2}) are in MPa.
3. Values of elastic moduli of concrete (E_{c1}, E_{c2}) are in GPa.

CHAPTER 3

ESTIMATION OF LOAD CARRYING CAPACITY AND REMAINING FATIGUE LIFE

3.1 Background

After the insitu material properties of the pavement structure have been estimated (as discussed in the previous chapter), the determination of the load carrying capacity is essentially a design procedure only in reverse. It involves, arriving at the permissible load depending upon the insitu flexural strength of the concrete and the stresses induced due to the actual loading of the pavement. For this purpose the most widely used model is based on Westergaard's analysis and his modified formulas for computing stresses in concrete pavement of airfields [9]. The Indian Road Congress (IRC) recommends the same formulas and so does the International Civil Aviation Organization (ICAO) [13]. The formulas for the stresses developed under the interior and edge loading are given below in eqs. 3.1 and 3.2.

$$\sigma_i = 0.275(1+\nu_c)\frac{P}{h^2}\left[\log_{10}\left(\frac{E_c h^3}{kb^4}\right) - 0.35632\right] \qquad (3.1)$$

$$\sigma_e = 0.529(1+0.54\nu_c)\frac{P}{h^2}\left[\log_{10}\left(\frac{E_c h^3}{kb^4}\right) - 0.71\right] \qquad (3.2)$$

where, h = thickness of the concrete slab

$b = (1.6c^2+h^2)^{0.5} - 0.675$ when $c \leq 1.724h$

$b = c$ when $c > 1.724h$

c = radius of the circular area over which the load P is assumed to be uniformly distributed.

As the stresses induced due to edge loading are critical eq.3.2 is normally adopted for the computation of stresses [9]. Even though in the above formulation, pavements are assumed to be semi-infinite, in actual practice, the adjacent panels are 'joined' at the edges (as shown in Fig.2.6). To account for this, it is assumed that when the load is placed at an edge of a slab, one-quarter of the load is transferred to the adjacent slab. The remaining load (0.75P) is increased by 30 percent to take care of the curling stresses due to temperature variations and fluctuations in the moisture content of the subgrade and other contingencies [12]. The net load ($0.75 \times 1.3 = 0.98$ P) is to be actually used in the Westergaard's edge load formula. As $0.98P \simeq P$, for simplicity the eq.3.2 has been adopted as it is in the present work.

3.2 Method Currently Used

In literature the structural capacity of a pavement is denoted in terms of Pavement Classification Number (PCN), which is an index rating, 1/500 of the permissible load in kg that can be borne by the pavement when applied by a standard single wheel of 1.25 MPa tire pressure. For example a PCN rating of 30 would ıean the permissible load (single wheel of 1.25 MPa tire ıressure) that can be borne by the pavement is 15000 kg. To

arrive at the PCN, first the maximum allowable stress is found out by dividing the modulus of rupture of the insitu concrete by a factor of safety, for which the Portland Cement Association (PCA) suggests the values from 1.25 to 1.5 [11]. Then the permissible load is determined and defined as 'that load which when applied to the pavement results in inducing the maximum tensile stress, equal to the maximum allowable stress'.

Whereas each runway is characterized by its PCN, the aircrafts are characterized by an Aircraft Classification Number (ACN). The ACN of an aircraft is numerically defined as two times the derived single wheel load of 1.25 MPa tire pressure expressed in thousand of Kgs. For example an ACN of 30 would mean that the derived single wheel load of that aircraft is 15000 Kgs.

In the final analysis the PCN and ACN are compared (Fig. 3.1) and the pavement is declared fit to service any aircraft of ACN ≤ it's PCN. Thus it is clearly seen that even though the loads are essentially repetitive in nature, the evaluation method do not incorporate the concept of fatigue life explicitly but only in empirical form in terms of a 'safety factor'.

3.3 Proposed Method

To arrive at a rational method for structural evaluation of a concrete pavement, it is not sufficient to consider the magnitude of the load alone but also the number of repetitions expected at different load levels. Obviously, a pavement can support a larger load for fewer repetitions and a smaller load for a greater number of repetitions. In other words,

the load carrying capacity of a pavement will be low for pavements with short residual life and high for those with long residual life.

Therefore it is imperative that the structural evaluation of pavement includes not only the magnitude of maximum permissible load but also the residual fatigue life taking into account the number of load repetitions. The details of the model proposed to incorporate the concept of repeated application of a load (level) into the runway evaluation process is presented in this chapter.

3.4 Development of Methodology For Estimation of Remaining Fatigue Life

As part of structural evaluation of a pavement at a particular time, it is important to know about it's remaining fatigue life under a particular set of loading and annual repetitions. Prediction of pavement's residual fatigue life would essentially involve :

(1) determination of stresses induced due to actual loading,

(2) calculation of allowable air traffic loading,

(3) computation of fatigue damage due to actual repetitions and

(4) finally estimation of the fatigue life.

The stresses imposed due to actual loading can be determined by eq.3.2. To arrive at other parameters mentioned above it is essential to understand the behaviour of concrete under fatigue loading.

3.4.1 Fatigue of concrete

Various studies on the fatigue properties of concrete show that an induced flexural stress could be repeated indefinitely without causing rupture, provided the intensity of extreme fibre stress does not exceed approximately 50 percent of the modulus of rupture [11]. And if the stress ratio was above 50 percent, the allowable number of stress repetitions to cause failures decreased drastically as the stress ratio increased. Although the arbitrary use of 50 percent stress ratio as a dividing line has not actually been established, this assumption has been used most frequently as basis for concrete pavement design [11]. Also a number of equations are available in literature which relate the load levels to the cycles to failure (S-N curves). These are based on fatigue tests carried out on test and in service pavements of airfields by US Army Corps of Engineers (USACE), American Association of State Highways and Transportation Officials (AASHTO) and Portland Cement Association (PCA). The following equations are recommended by the PCA (Packard and Tayabji, 1985) :

$$\text{For } \sigma/S_c \geq 0.55 \quad : \log N_f = 11.737-12.077(\sigma/S_c) \tag{3.3}$$

$$0.45 < \sigma/S_c < 0.55 : N_f = (4.25777/(\sigma/S_c - 0.4325))^{3.268} \tag{3.4}$$

$$\text{For } \sigma/S_c \leq 0.45 \quad : N_f = \text{unlimited} \tag{3.5}$$

where, σ is the maximum flexural stress induced in the concrete

slab due to actual loading, S_c is the modulus of rupture of concrete and N_f is the allowable number of repetitions. The plot of stress ratio versus number of repetitions to failure, based on eqs. 3.3 through 3.5 is shown in Fig. 3.1a. It can be seen that PCA's method assumes a stress ratio of 0.45, below which no fatigue damage need be considered and is more conservative than the AASHTO method.

3.4.2 Cracking Index

Assuming that the allowable number of repetitions (cycles to failure) and the actual number of repetitions (per annum) at a load level are N_f and n_i, the total damage to the runway used by 'm' types of aircrafts (loads) at the end of an year can then be computed using the Miner's rule and is represented as a cracking index (CI) defined below

$$CI = \sum_{i=1}^{m} \frac{n_i}{N_f} \tag{3.6}$$

3.4.3 Computation of fatigue life

The total fatigue life of a pavement is thus the inverse of the CI per annum computed in eq. 3.6. However, in the present study two procedures to carry out this analysis have been examined. In the first procedure the design aircraft is considered as the basis of calculation as is done in designing. In the second procedure the entire spectrum of aircraft with their corresponding annual repetitions are taken into account.

Procedure 1.

In this the fatigue life is determined by computing the CI with respect to the design aircraft and involves following steps.

(a) Calculation of insitu flexural strength of the concrete using the following relationship as given in IS Code-456

$$S_c = \frac{0.7 \times E_c}{5700} \tag{3.7}$$

Note. The E_c as estimated from FWD test may be used (chapter 2)

(b) Determination of actual stresses induced due to design aircraft loading using Westergaard's edge load formula, eq. 3.2.

(c) Finding the stress ratio and then evaluating the allowable number of load repetitions, N_f (using eqs.3.3 through 3.5).

(d) Calculating CI depending upon the actual repetitions, n in terms of the design aircraft in an year.

(e) Finally estimation of fatigue life, Y_f as

$$Y_f = \frac{1}{CI} = \frac{1}{n/N_f} \tag{3.8a}$$

Procedure 2.

In this procedure the steps for arriving at the CI value remain same as outlined above. However, the data for each of the aircraft is considered separately to compute the induced flexural stress, the stress ratio, allowable number of load repetitions and the corresponding CI. The CI for each aircraft case is then

summed up to arrive at the cumulative CI value per annum. The fatigue life in this case can then be given by

$$Y_f = \frac{1}{CI} \tag{3.8b}$$

where, CI is calculated as per eq. 3.6.

On comparing procedures 1 and 2 it has been found that the later is more rational in estimating the fatigue life of a pavement. This has been discussed in greater detail in section 3.5.1.

3.4.4 Computation of Remaining Fatigue Life

The remaining fatigue life of a pavement which is in service for a given number of years can be estimated by extending the approach adopted above in determination of fatigue life. For example to know the residual fatigue life after Y years of pavement service life, based on actual aircraft traffic statistical data for each year, CI for each year can be computed as explained in section 3.4.3. Then the Average Annual Cracking Index (AACI) after Y years can be given by :

$$AACI = \sum_{i=1}^{y} \frac{CI_i}{Y} \tag{3.9}$$

The remaining fatigue life, Y_{rf} in years of the pavement can then be calculated by :

$$Y_{rf} = (1/AACI - Y) \tag{3.10}$$

It may be noted that the remaining fatigue life too can be estimated by both the procedures as outlined in section 3.4.3.

The overall methodology thus developed for estimating remaining fatigue life of a concrete pavement is presented in Fig. 3.2.

3.5 Illustrative Example

The methodology for estimation of remaining fatigue life of a pavement as developed above can be further illustrated through an example. A design example has been taken from the ICAO manual [13] and is reproduced in Table 3.1. In the design procedure a design aircraft is first arrived at and then the anticipated annual repetitions of various aircrafts are converted to equivalent annual repetitions in terms of the design aircraft. The design thickness is then based on the design aircraft load and its equivalent annual repetitions. For this example, taking arbitrarily the concrete flexural strength as 4.5 MN/m^2 and k value of 80 MN/m^3, the resulting design thickness for 16000 annual repetitions is 400 mm. Now provided that the anticipated annual repetitions remain unchanged the fatigue life of the pavement should be 20 years as suggested in the manual.

Now assuming that the pavement in the first year of its service life has been subjected to the same traffic as was assumed for the design purpose, analysis based on cumulative damage due to fatigue is carried out by procedures 1 and 2.

Procedure 1.

In this procedure the design aircraft is considered as the basis of calculation as is done in the design procedure. The fatigue life is calculated by determining the cracking index with respect to the design aircraft. As the pavement is subjected to 16000 load repetitions of the design aircraft, the cracking index at the end of first year will be $16000/N_f$. Going through the steps as enumerated in section 3.4.4 for computing induced flexural stress, stress ratio, N_f and CI, the results obtained are (Table 3.2) :

Stress Ratio = 0.5214

CI = 0.0517

Fatigue life in years = 19.64

Therefore, the fatigue life of the pavement is 19.64 years (the inverse of the CI), and is same as the design life of 20 years. This shows that the concept of fatigue analysis can be applied to asses the life of a pavement and in fact can be used to verify the design arrived at.

Procedure 2.

In this procedure the data for each of the eight aircrafts is considered separately to compute the induced flexural stress, the stress ratio, allowable number of repetitions and the CI. The same is tabulated in Table 3.3. The cumulative CI obtained at the end of first year of service life is 0.0406. This results in the fatigue life of 24.6 years, which is approximately 4 years more than that worked out by procedure 1 or the design life.

3.5.1 Comparison of Procedures

The difference in the computed fatigue life in the above two procedures is apparent as the stress ratio in procedure 1 remains constant and is in terms of the design aircraft. However for different aircrafts the stress ratio would be different and since allowable number of repetitions take a quantum jump when the stress ratio is 0.45 or below, the aircrafts in this category can carry out unlimited number of repetitions without affecting the fatigue life. Therefore in reality the fatigue life of the example pavement is more than 20 years. This shows that the procedure 1 is conservative and procedure **2** is more rational method of determining the fatigue life of a pavement.

3.5.2 Prediction of Remaining Fatigue Life

For the same example pavement, the remaining fatigue life is now computed after five years of service life by both the procedures. The annual repetitions of the aircraft were arbitrarily altered for five years. The cumulative CI, AACI and the residual fatigue life arrived at in both the procedures are tabulated in Table 3.4.

It can be observed that after five years of service life (15 years of life still remaining) while procedure 1 has resulted in a residual life of 13 years, the procedure 2 has resulted in 19 years. This further brings out the fact that procedure 1 is conservative in estimating the remaining fatigue

life and procedure 2, which is a rational method of evaluating fatigue life, is therefore recommended.

3.6 Relationship Between Load Carrying Capacity and Remaining Fatigue life

From the above it is clear that at any point of time, during its service life, the load carrying capacity of a pavement will depend upon the remaining fatigue life desired. If the load repetitions are kept constant, then at lower magnitude of load the remaining fatigue life is more and vice-e-versa. Therefore it is important to establish a relationship between the load magnitude, the number of load applications or repetitions and the remaining fatigue life.

To establish the above relationship a study on remaining fatigue life has been carried out at different aircraft loads and their annual repetitions. The representative values of E_m and E_c for the pavement have been arbitrarily assumed. In actual these would be arrived at by carrying out statistical analysis of the estimated values of E_m and E_c for each slab of the pavement as indicated in chapter 2. The results of the study are shown in Fig.3.2.

It can be seen that the remaining fatigue life has a logarithmic variation with respect to the load magnitude. Also for attaining a certain fatigue life, if the number of repetitions are increased, the magnitude of the load which can be allowed to operate on an airport pavement decreases. This clearly brings out that it is not sufficient to classify the pavements only in terms of the PCN or the maximum permissible load.

3.7 Proposal

From the above study it is suggested that after estimation of the insitu elastic moduli of concrete and subgrade by FWD testing, an evaluation plot as shown in Fig.3.2 for the pavement be prepared. Then, depending upon the fatigue life desired and the expected numbers of annual repetitions in future, the load carrying capacity of the pavement be arrived at. This plot would also indicate if strengthening of the pavement is required to cater for a particular heavier load operating aircraft.

3.8 Summary

This chapter describes the methodology developed for estimation of load carrying capacity in terms of residual fatigue life desired, for the pavement under evaluation. The important points discussed in the chapter are summarized below :

1. Present evaluation methods incorporate the fatigue of pavement due to repetitive loading only in empirical form in terms of a 'safety factor'. Therefore a simplified method based on cumulative damage due to fatigue of the pavement has been proposed.
2. The methodology developed involves computation of stresses induced in the pavement due to actual loading, determination of stress ratio, finding out the allowable number of load applications from the S-N curve (plot of stress ratio versus number of repetitions to failure), computation of cumulative fatigue damage as 'cracking

index' and finally estimating the remaining fatigue life. Though the discussion is confined to the S-N curve given by PCA, the approach is applicable for any S-N curve of concrete pavement.

3. It is realized that the residual (remaining) fatigue life of the pavement is dependent on the magnitude of loads and the frequencies thereof in the future. Therefore, the present system of mentioning a single PCN value arrived at by evaluating the maximum permissible load, as an indicator of pavement's structural capacity needs modification.

4. It is found that considering the load and repetitions only in terms of the design aircraft, the remaining fatigue life obtained is much less than what is obtained by considering the entire spectrum of aircrafts and their corresponding annual repetitions. Therefore, it is proposed that the actual air traffic may be used in carrying out the fatigue analysis to arrive at more reasonable assessment of the remaining fatigue life.

5. It is suggested that, based on the insitu elastic moduli of the subgrade and the concrete slab estimated by FWD tests, a plot of of load level versus fatigue life at varying number of repetitions be prepared. Then based on future traffic spectrum and fatigue life desired the structural capacity of the pavement be fixed.

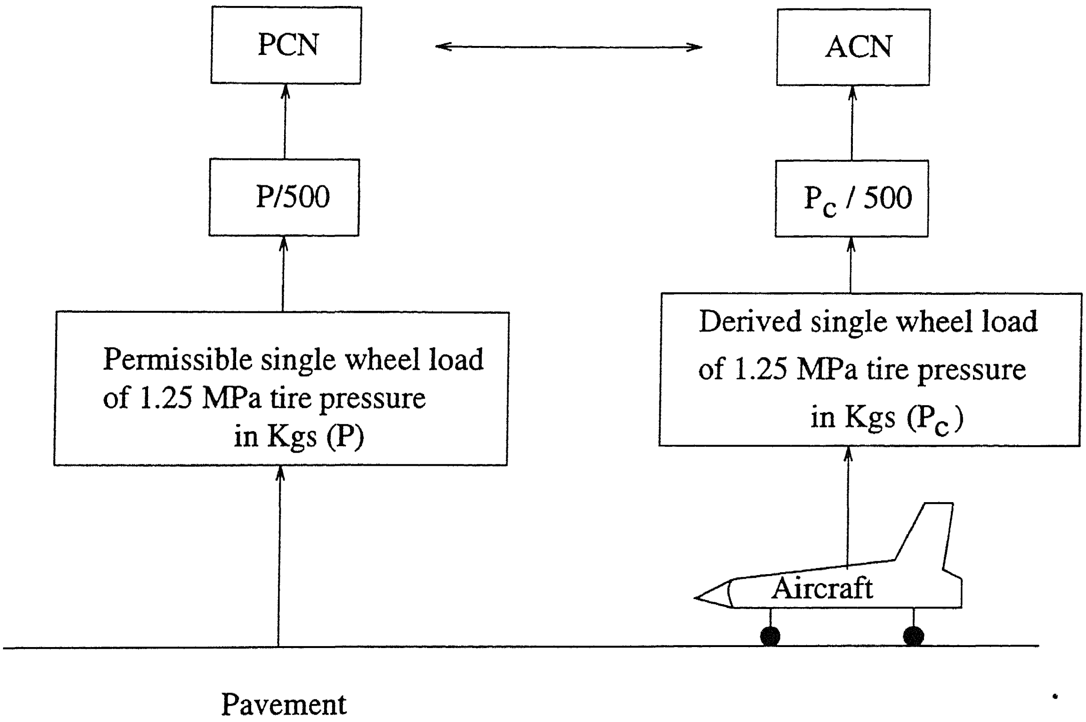


Fig. 3.1 ACN - PCN method of pavement's structural classification

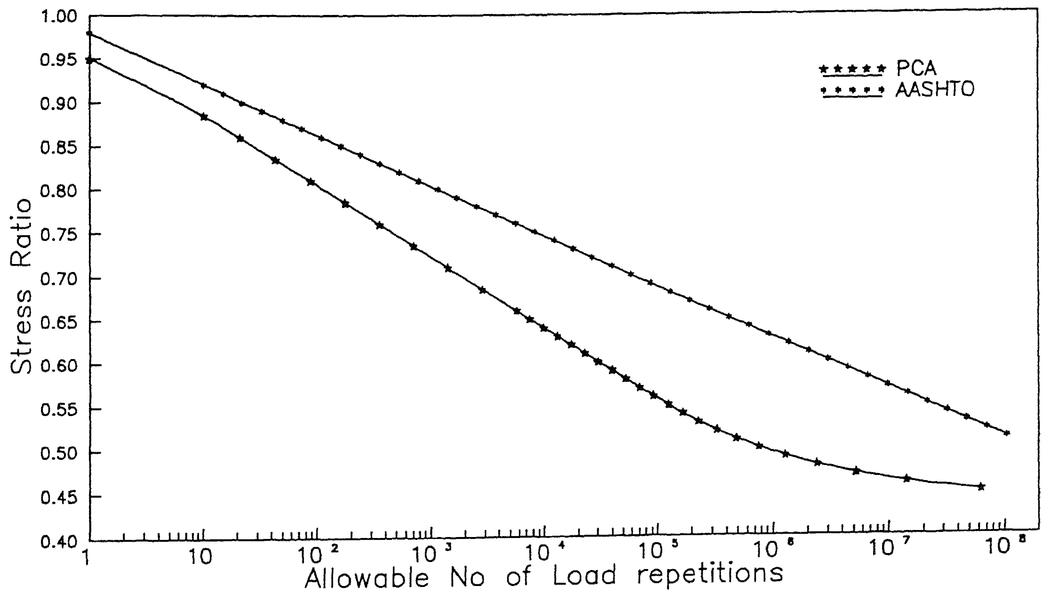


Fig. 3.1a Plot of stress ratio versus number of repetitions to failure

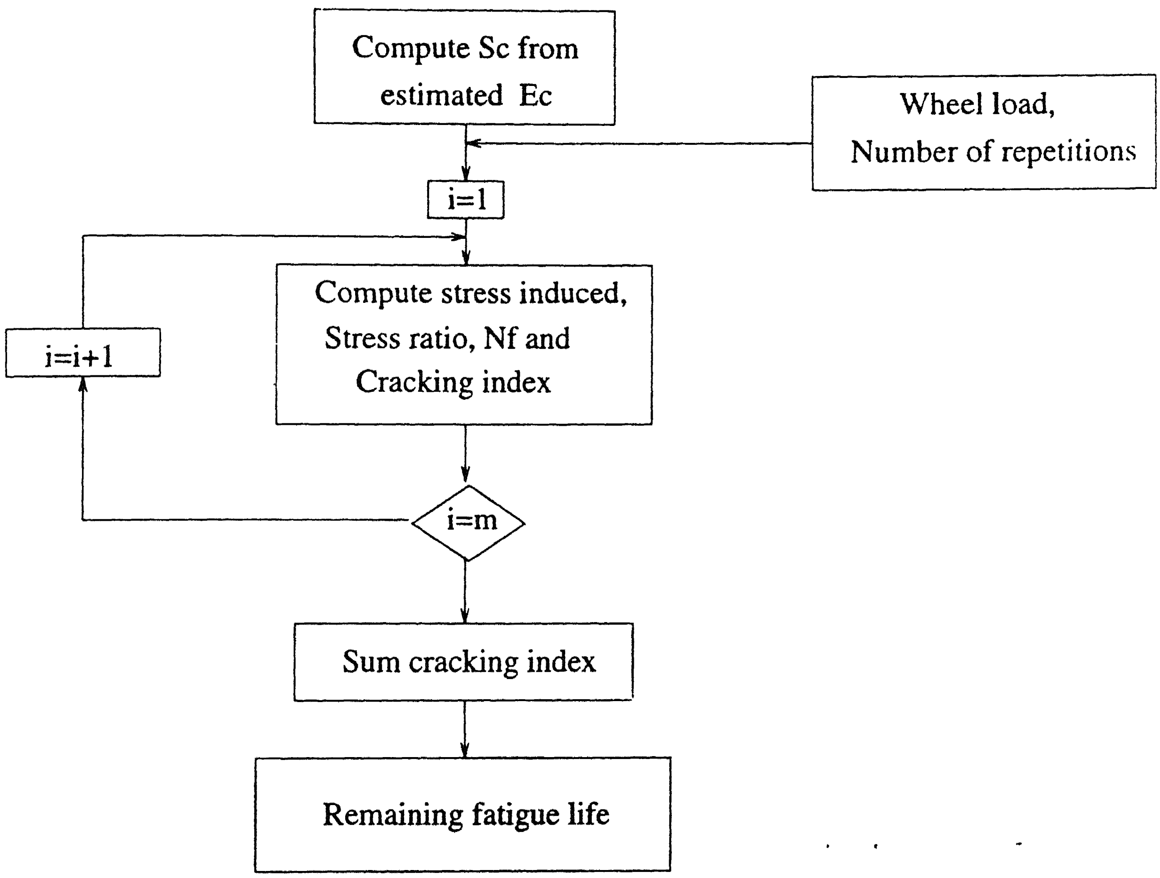
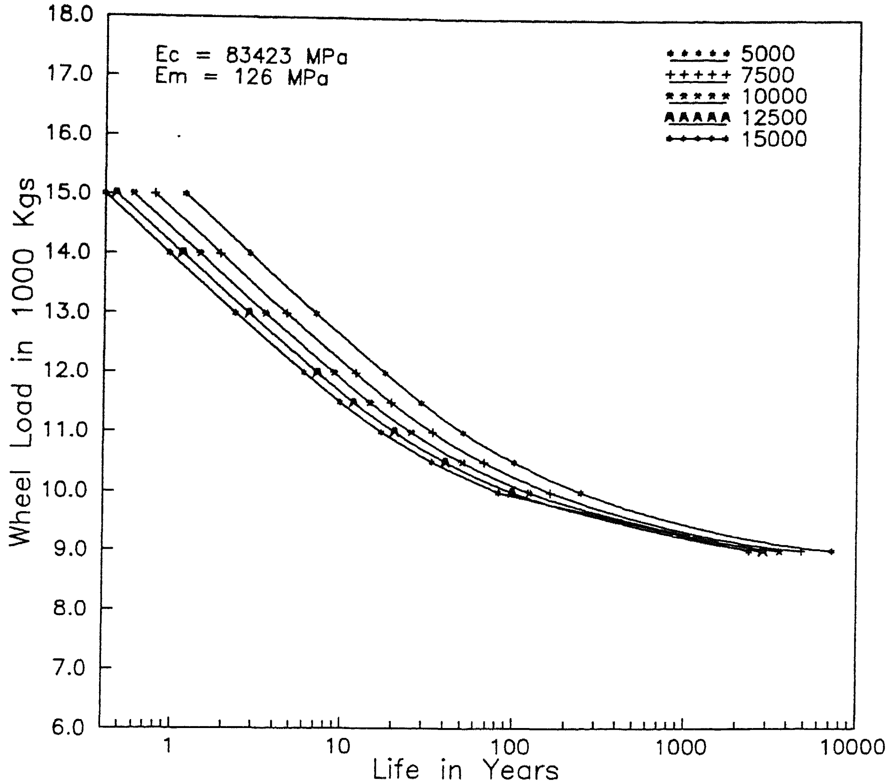
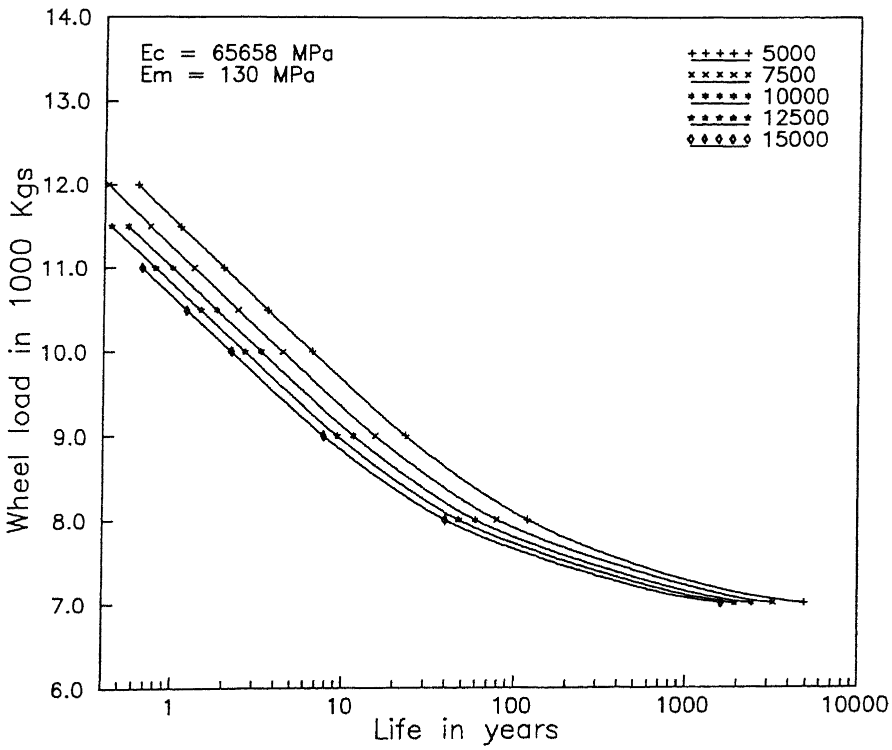


Fig. 3.2 Flow chart for calculating remaining fatigue life



(a)



(b)

Fig. 3.3 Evaluation of concrete airport pavement in terms of load levels, number of load repetitions and fatigue life

Table 3.1 Design Example

Aircraft type	Wheel load	Annual repetitions	Equivalent annual repetitions
727-100	17240	3760	1891
727-200*	20520	9080	9080
707-320B	17610	3050	2764
DC-9-30	11630	5800	682
CV-880	9940	400	94
737-200	12440	2650	463
L-1011-100	16160	1710	83
747-100	16160	85	1184
			Total = 16241

* - Design Aircraft ; Source - ICAO Manual, Part 3, 1983.

Table 3.2 Estimation of fatigue life by considering the loads in terms of the design aircraft for computation of cracking index. (Procedure # 1)

Design aircraft wheel load	Equivalent annual repetitions	Stress ratio	CI	Life in years
20,500 Kg	16241	0.5214	0.0517	19.64

Table 3.3 Estimation of fatigue life by considering each aircraft separately for computation of cracking index. (Procedure # 2)

Aircraft type	Wheel load	Annual repetitions	Stress ratio	N_f	CI	ΣCI
727-100	17240	3760	0.46	1.43E07	0.0003	0.0003
727-200	20520	9080	0.52	2.29E05	0.0396	0.0399
707-320	17610	3050	0.47	4.25E06	0.0007	0.0406
DC-9-30	11630	5800	0.33	Unlimited	-	do
CV-880	9940	400	0.29	do	do	do
737-200	12440	2650	0.35	do	do	do
L-1011	16160	1710	0.44	do	do	do
747-100	16160	85	0.46	1.43E07	do	do
						0.0406

Life = 1/CI = 24.63 Years

Table 3.4 Comparision of Remaining Fatigue Life

Procedure # 1		Procedure # 2	
Year	CI	Year	CI
1	0.0517	1	0.0406
2	0.0514	2	0.0380
3	0.0511	3	0.0377
4	0.0528	4	0.0411
5	0.0568	5	0.0432

Procedure # 1:

$$\sum_{y=1}^{5} CI = 0.2633$$

AACI = 0.05266

Residual life = 13 YEARS

Procedure # 2:

$$\sum_{y=1}^{5} \sum_{m=1}^{8} CI = 0.2006$$

AACI = 0.0401

Residual life = 9 Years

CHAPTER 4

CONCLUSIONS

4.1 Conclusions

In this thesis a model for structural evaluation of concrete runways on the basis of non-destructive testing by Falling Weight Deflectometer (FWD) has been presented. It includes estimation of insitu elastic moduli of concrete slab and subgrade. Then, using these moduli and carrying out analysis based on cumulative damage due to fatigue, a new method has been proposed to classify the structural capacity of the pavement in terms of load levels, number of load applications and remaining fatigue life desired. The following conclusions have been obtained :

1. The insitu elastic moduli of subgrade can be estimated by converting the pavement structure into the subgrade material using the method of equivalent thickness and then applying Boussinesq's deflection equation.
2. The equivalent thickness of a concrete slab linearly increases with its real thickness. The magnitude of increase varies from 6 times for the slab which is 150 mm thick to 8 times for a slab which is 300 mm thick. The magnitude depends upon the relative stiffness of the concrete slab with respect to the subgrade.
3. The insitu concrete modulus of elasticity can be estimated

using the subgrade elastic modulus determined and then matching the measured deflection (obtained by FWD) with the computed deflection under the load. For computing deflections Woinowsky-Kreiger's analysis of elastic plate over Winkler's foundation has been found to be more suitable.

4. Using the estimated insitu moduli, the residual (remaining) fatigue life of the pavement under evaluation can be estimated on the basis of cumulative fatigue damage due to repetitive loading of the pavement. The S-N curve and Miner's rule are used for the purpose. In this study the S-N curve proposed by the Portland Cement Association has been used.
5. The residual life of the pavement is dependent on the load levels and their annual repetitions. It is confirmed that the residual life for high loads is low and vice - versa.
6. Estimation of fatigue life, considering the load and repetitions only in terms of the design aircraft, yields a more conservative value than considering the entire spectrum of aircrafts and their corresponding annual repetitions. The latter method being more rational is therefore proposed to make a reasonable assessment of the pavement's remaining fatigue life.
7. Based on analysis of fatigue life a plot of load levels versus fatigue life at varying repetitions, as illustrated in this thesis, can be prepared. This plot will provide information concerning :

   (a) pavement's load carrying capacity depending upon the

future number of annual load repetitions and fatigue life desired.

(b) strengthening of the pavement (requirement of overlay).

4.2 Application of Present Work

Though the commercial software available with the FWD equipment can analyze the deflections to estimate the moduli , it acts as a 'black box' providing no scope for scrutiny or the basis of computation. The present work affords an avenue for indigenous development of a software which may be PC based. This would obviate the requirement of analyzing the deflection data using only the microcomputer of the FWD device. This would also assist in carrying out an another FWD testing at the same time, thus saving valuable time.

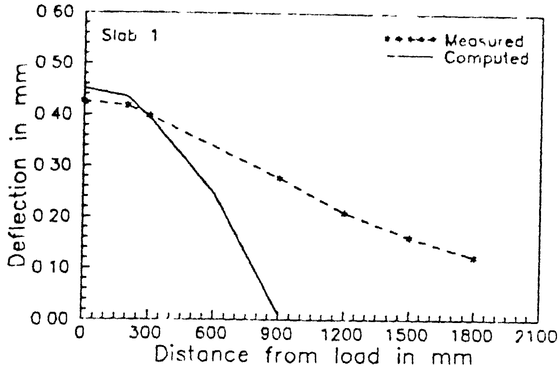
4.3 Scope for Future Studies

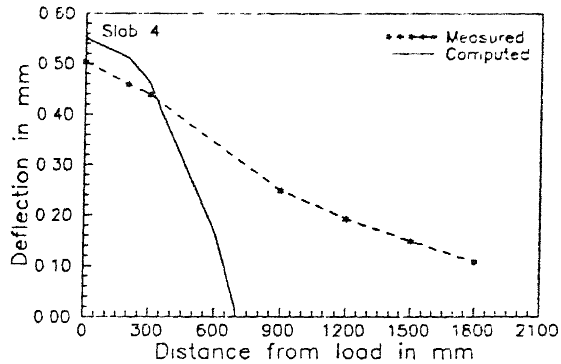
1. The model presented in the thesis can be further refined by carrying out similar studies for a wider range of slab thicknesses. The results of the model may be compared with the experimental findings, obtained directly by strength analysis of the core samples and plate loading tests on the subgrade.
2. The method proposed here can be used to directly estimate the remaining service life etc. depending upon the (instantaneous) values of elastic moduli of subgrade and

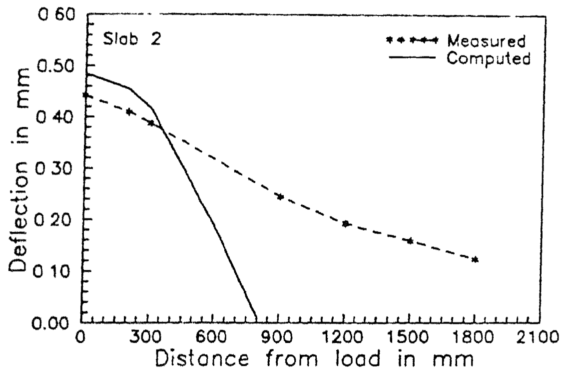
concrete. Thus, if the variation in these parameters over time (on account of deterioration or loading) is known it can be easily incorporated to obtain a more realistic estimate on service life.

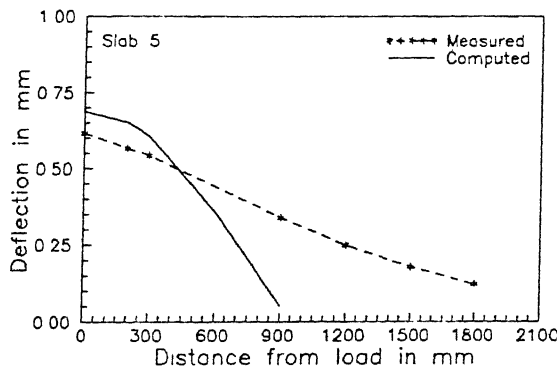
APPENDIX

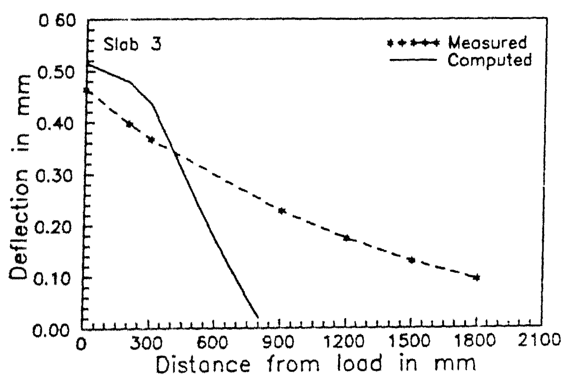
A1 Figures showing comparison of the estimated deflections using Westergaard's model with the actually obtained values for different slabs

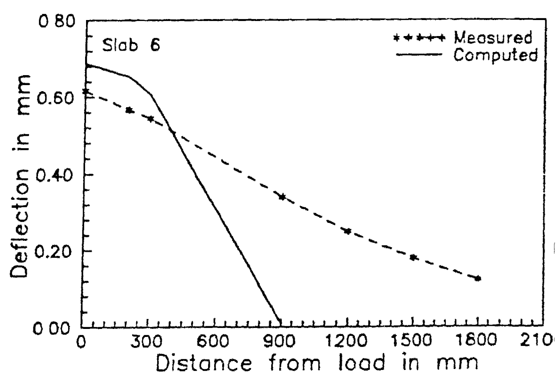


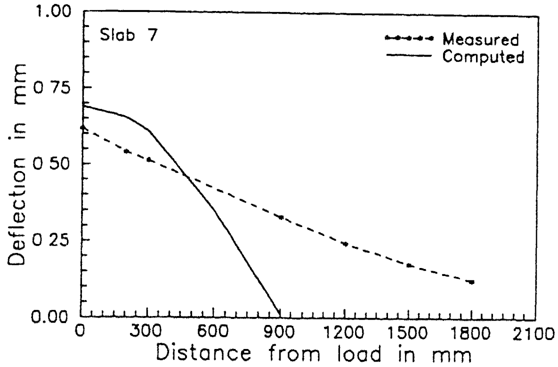


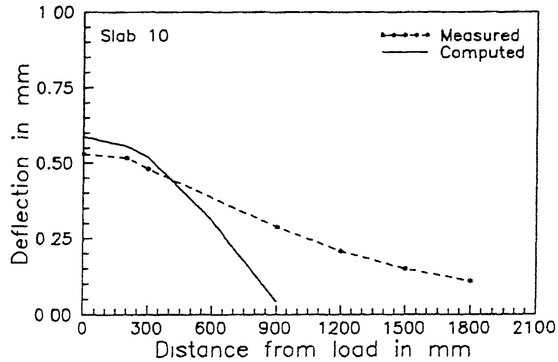


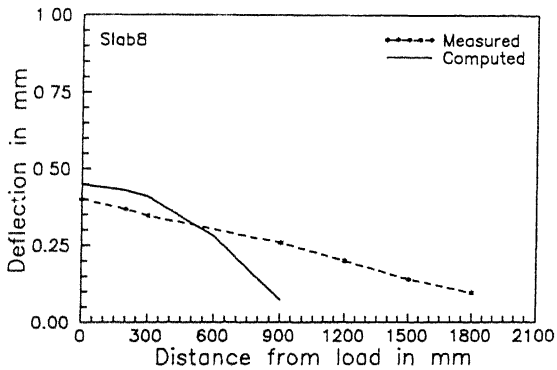


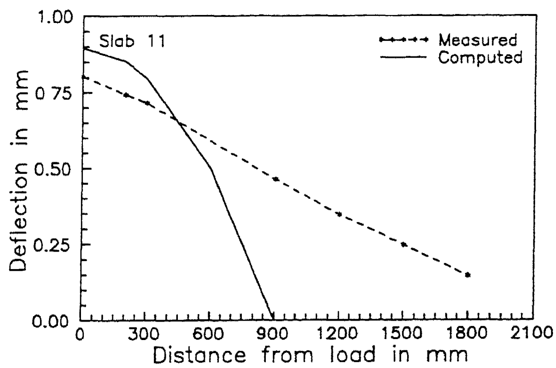


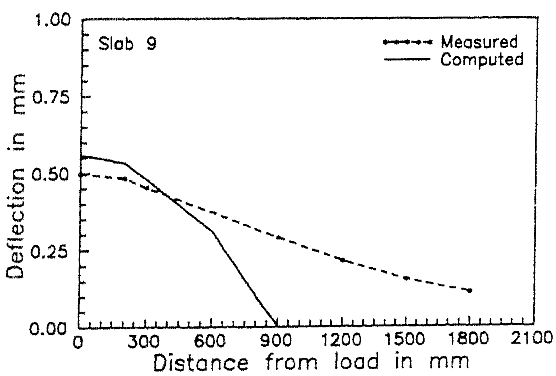


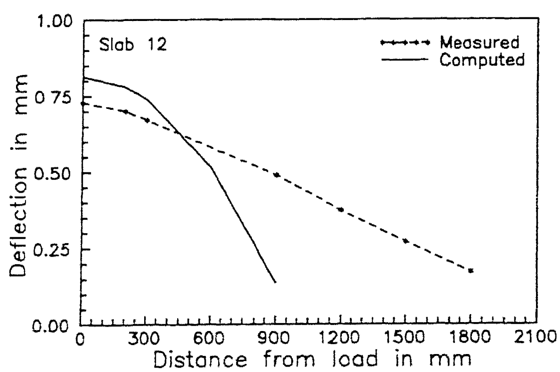

Slab 7
Measured
Computed
Deflection in mm
1.00
0.75
0.50
0.25
0.00
0
300
600
900
1200
1500
1800
2100
Distance from load in mm

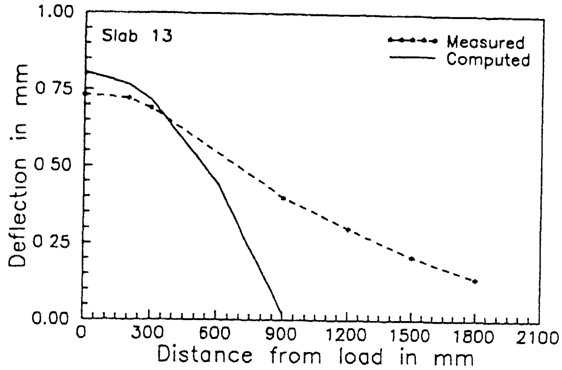

Slab 10
Measured
Computed
Deflection in mm
1.00
0.75
0.50
0.25
0.00
0
300
600
900
1200
1500
1800
2100
Distance from load in mm


Slab8
Measured
Computed
Deflection in mm
1.00
0.75
0.50
0.25
0.00
0
300
600
900
1200
1500
1800
2100
Distance from load in mm

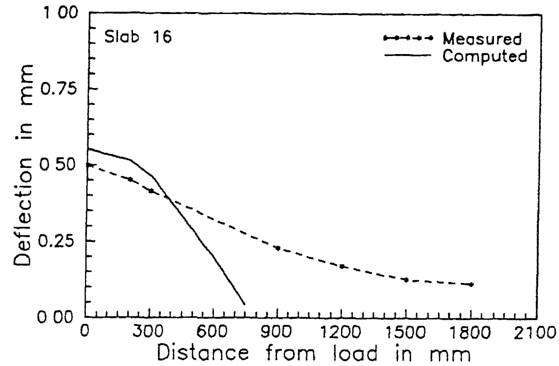

Slab 11
Measured
Computed
Deflection in mm
1.00
0.75
0.50
0.25
0.00
0
300
600
900
1200
1500
1800
2100
Distance from load in mm


Slab 9
Measured
Computed
Deflection in mm
1.00
0.75
0.50
0.25
0.00
0
300
600
900
1200
1500
1800
2100
Distance from load in mm

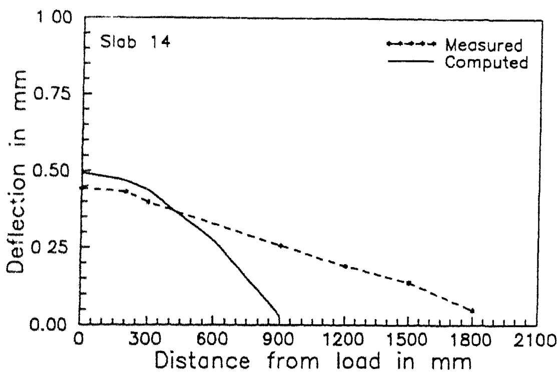

Slab 12
Measured
Computed
Deflection in mm
1.00
0.75
0.50
0.25
0.00
0
300
600
900
1200
1500
1800
2100
Distance from load in mm



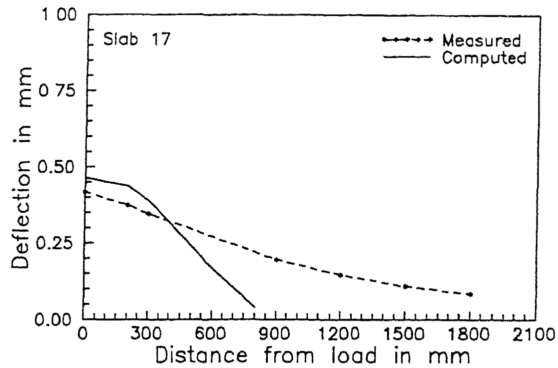
1.00
Slab 13
Measured
Computed
0.75
0.50
0.25
0.00
Deflection in mm
0
300
600
900
1200
1500
1800
2100
Distance from load in mm



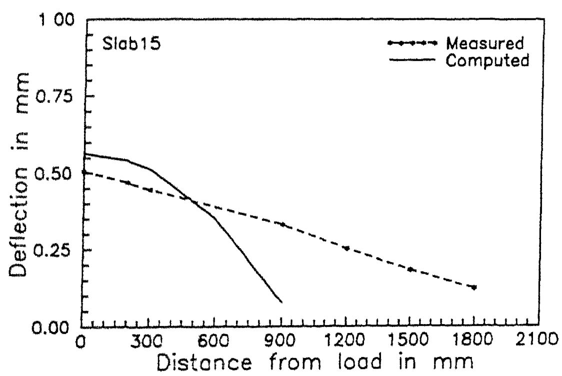
1.00
Slab 16
Measured
Computed
0.75
0.50
0.25
0.00
Deflection in mm
0
300
600
900
1200
1500
1800
2100
Distance from load in mm



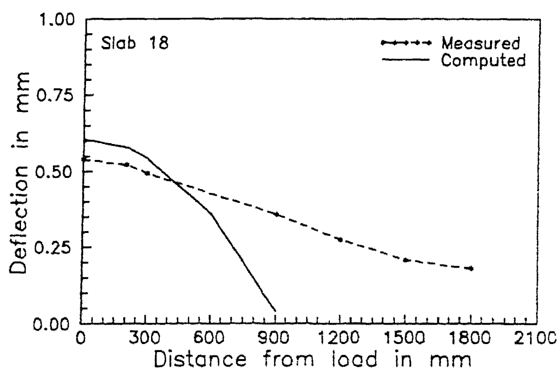
1.00
Slab 14
Measured
Computed
0.75
0.50
0.25
0.00
Deflection in mm
0
300
600
900
1200
1500
1800
2100
Distance from load in mm



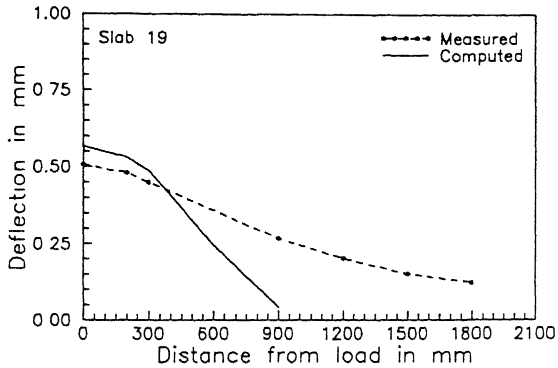
1.00
Slab 17
Measured
Computed
0.75
0.50
0.25
0.00
Deflection in mm
0
300
600
900
1200
1500
1800
2100
Distance from load in mm

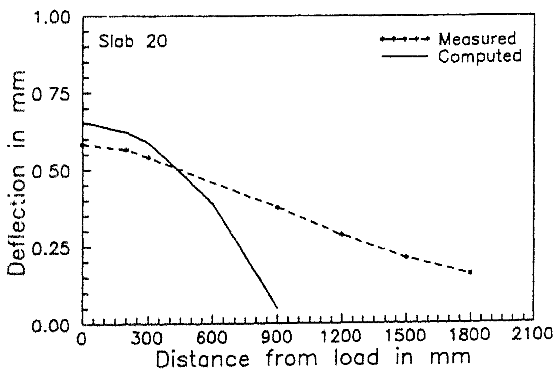


1.00
Slab15
Measured
Computed
0.75
0.50
0.25
0.00
Deflection in mm
0
300
600
900
1200
1500
1800
2100
Distance from load in mm

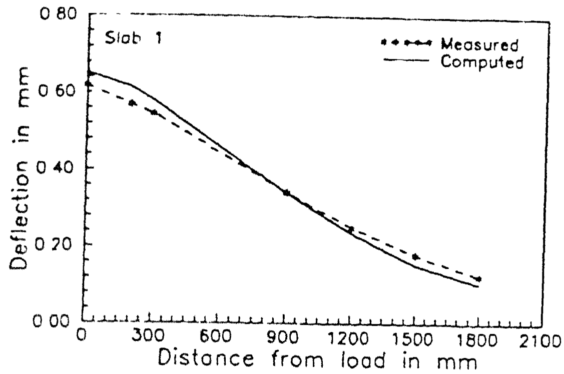


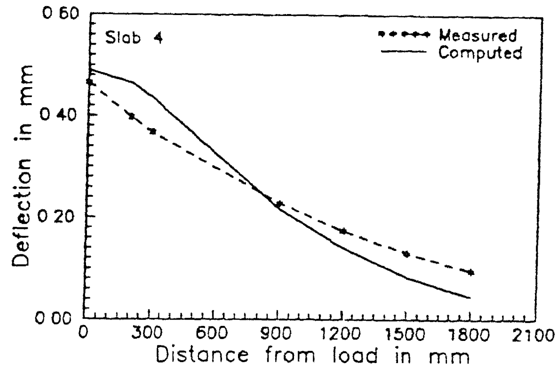
1.00
Slab 18
Measured
Computed
0.75
0.50
0.25
0.00
Deflection in mm
0
300
600
900
1200
1500
1800
2100
Distance from load in mm

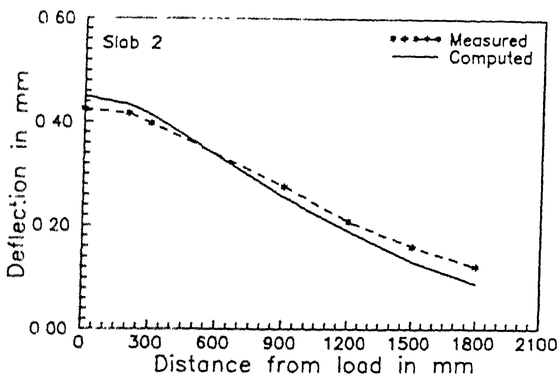

Slab 19
Measured
Computed
Deflection in mm
Distance from load in mm
1.00
0.75
0.50
0.25
0.00
0
300
600
900
1200
1500
1800
2100

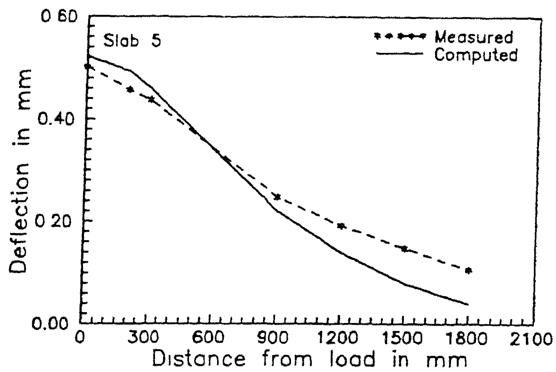

Slab 20
Measured
Computed
Deflection in mm
Distance from load in mm
1.00
0.75
0.50
0.25
0.00
0
300
600
900
1200
1500
1800
2100

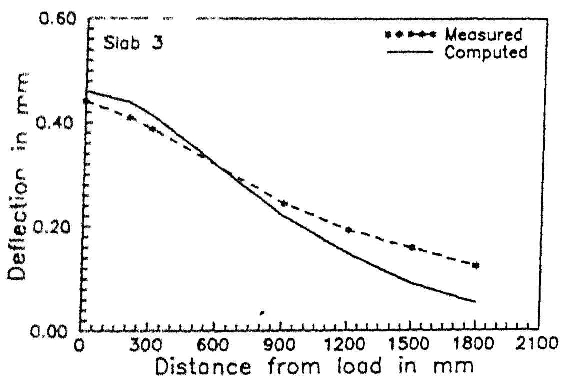
A2 Figures showing comparison of the estimated deflections using Woinowsky-Kreiger's model with the actually obtained values for different slabs.

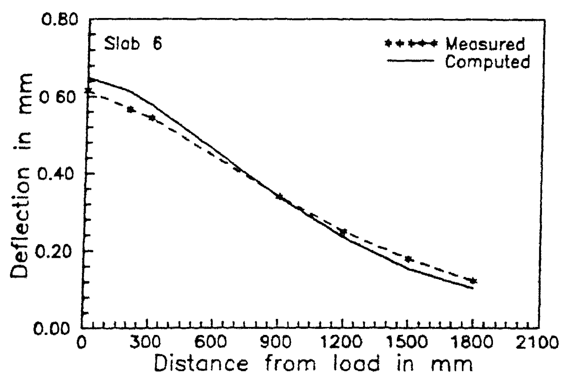


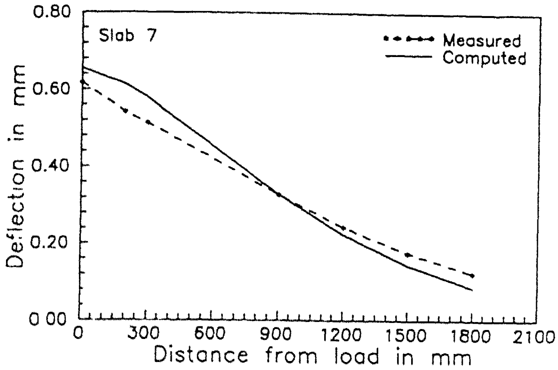


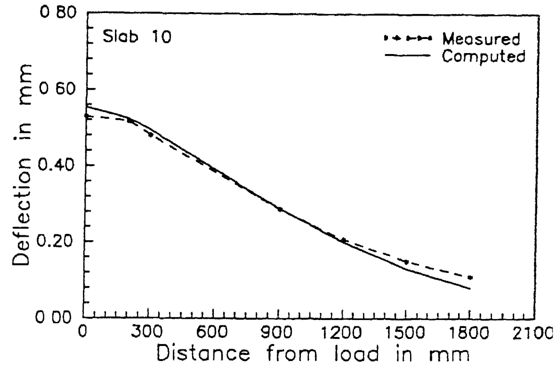


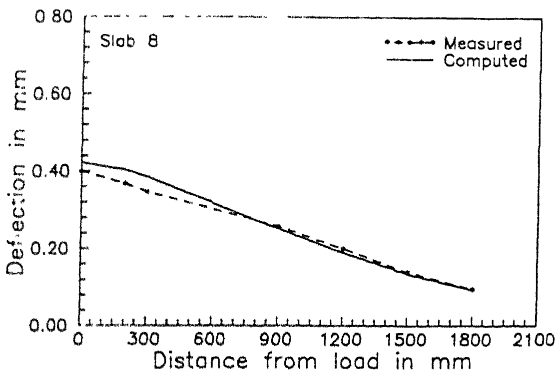


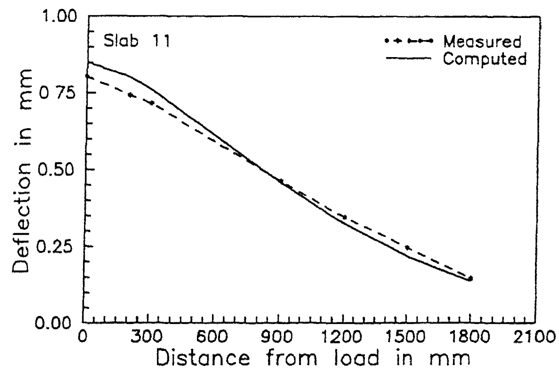


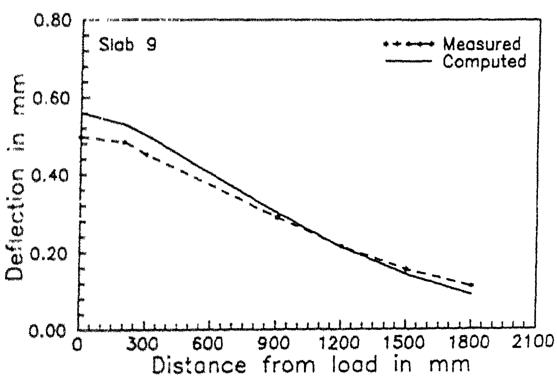


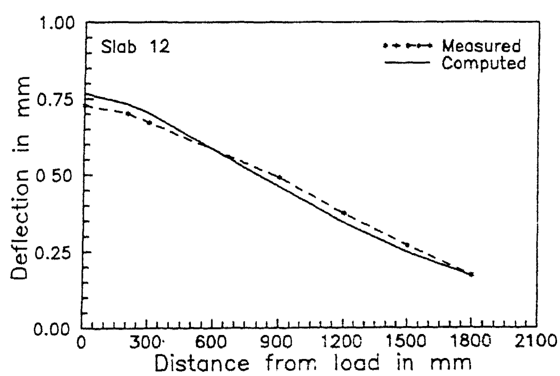

Slab 7
Measured
Computed
Deflection in mm
0.00
0.20
0.40
0.60
0.80
0
300
600
900
1200
1500
1800
2100
Distance from load in mm

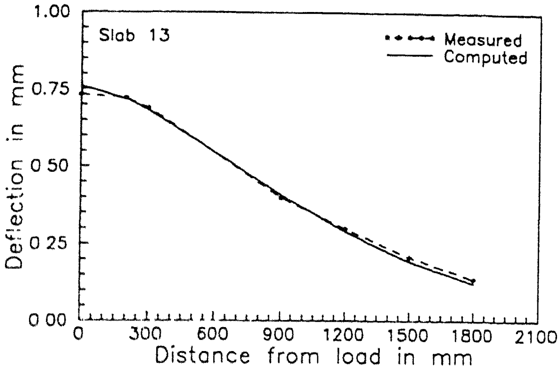

Slab 10
Measured
Computed
Deflection in mm
0.00
0.20
0.40
0.60
0.80
0
300
600
900
1200
1500
1800
2100
Distance from load in mm

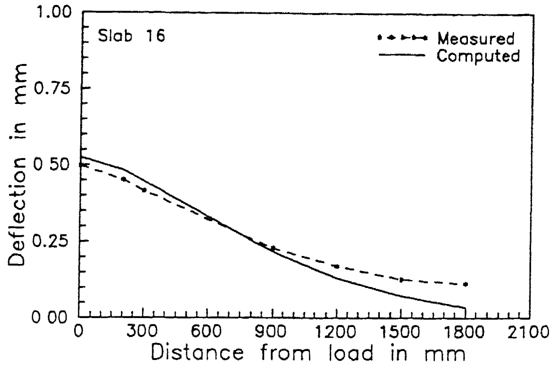

Slab 8
Measured
Computed
Deflection in mm
0.00
0.20
0.40
0.60
0.80
0
300
600
900
1200
1500
1800
2100
Distance from load in mm

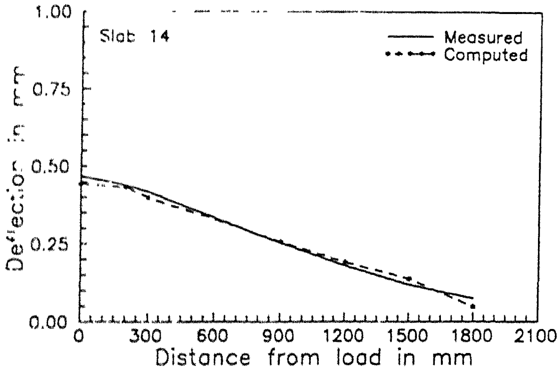

Slab 11
Measured
Computed
Deflection in mm
0.00
0.25
0.50
0.75
1.00
0
300
600
900
1200
1500
1800
2100
Distance from load in mm

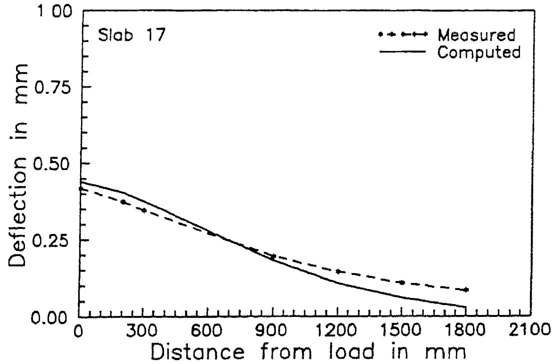

Slab 9
Measured
Computed
Deflection in mm
0.00
0.20
0.40
0.60
0.80
0
300
600
900
1200
1500
1800
2100
Distance from load in mm

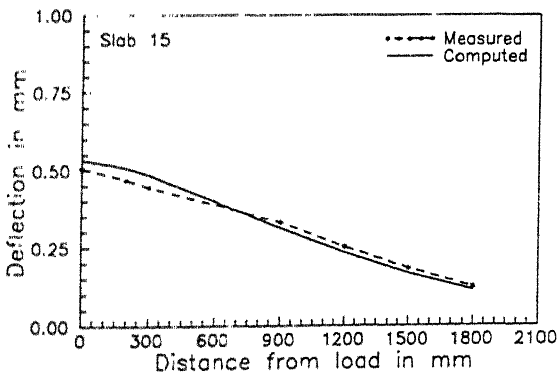

Slab 12
Measured
Computed
Deflection in mm
0.00
0.25
0.50
0.75
1.00
0
300
600
900
1200
1500
1800
2100
Distance from load in mm

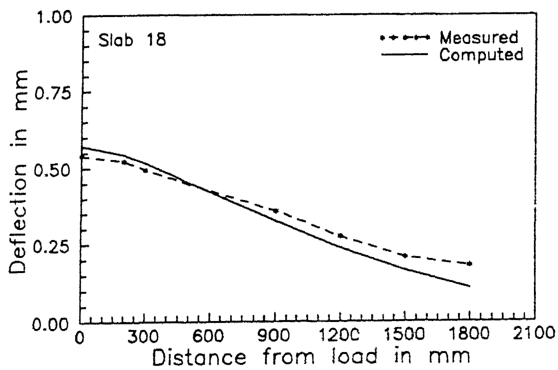

Slab 13
Measured
Computed
Deflection in mm
1.00
0.75
0.50
0.25
0.00
0
300
600
900
1200
1500
1800
2100
Distance from load in mm

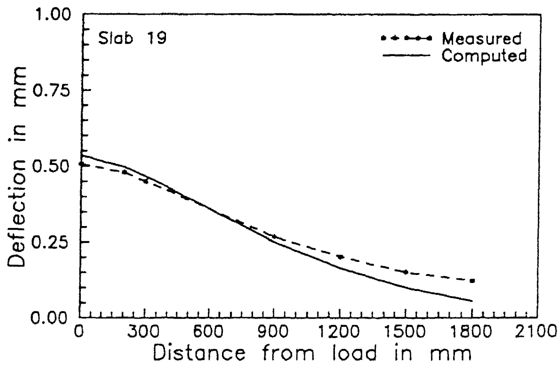

Slab 16
Measured
Computed
Deflection in mm
1.00
0.75
0.50
0.25
0.00
0
300
600
900
1200
1500
1800
2100
Distance from load in mm

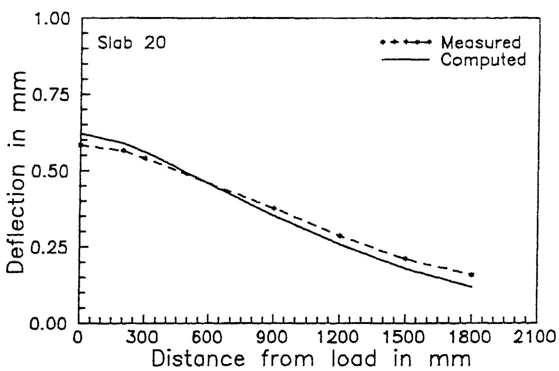

Slab 14
Measured
Computed
Deflection in mm
1.00
0.75
0.50
0.25
0.00
0
300
600
900
1200
1500
1800
2100
Distance from load in mm


Slab 17
Measured
Computed
Deflection in mm
1.00
0.75
0.50
0.25
0.00
0
300
600
900
1200
1500
1800
2100
Distance from load in mm


Slab 15
Measured
Computed
Deflection in mm
1.00
0.75
0.50
0.25
0.00
0
300
600
900
1200
1500
1800
2100
Distance from load in mm


Slab 18
Measured
Computed
Deflection in mm
1.00
0.75
0.50
0.25
0.00
0
300
600
900
1200
1500
1800
2100
Distance from load in mm


1.00
0.75
0.50
0.25
0.00
Slab 19
Measured
Computed
Deflection in mm
0
300
600
900
1200
1500
1800
2100
Distance from load in mm


1.00
0.75
0.50
0.25
0.00
Slab 20
Measured
Computed
Deflection in mm
0
300
600
900
1200
1500
1800
2100
Distance from load in mm

REFRENCES

[1] **Huang, Y.H.** Pavement Analysis and Design. Prentice- Hall, 1993.

[2] **Huang, Y.H. and X.Deng,** *"Finite element analysis of jointed concrete pavements,* Journal of Transportation Engineering, ASCE, Vol. 109, No. 5, PP, 689-705, 1990.

[3] **Boming Tang and X.Deng,** *"Application of FWD to structural evaluation of jointed concrete airfield pavements",* Third International Conference on Bearing Capacity of Roads and Airfields, 1990.

[4] **Ullidtz, P. and Peattie** *"Pavement analysis by programmable calculators",* Transportation Engineering Journal of the ASCE, Vol. 106 No.TES, Sept. 1980.

[5] **Scott, R.F.** Foundation Analysis, Prentice-Hall, 1981.

[6] ———. Principles of Soil Mechanics. Addison-Wesley Publishing Company, 1965.

[7] **Westergaard, H. M.** *" Stress concentrations in plates loaded over small areas",* Trans, ASCE, Vol 108, PP, 834-884, 1943.

[8] ———.*"Stresses in concrete runways of airports",* Procedings Highway Research Board, Vol 19, PP, 197-205, 1939.

[9] ———.*"New formulas for stresses in concrete pavements of airfields",* Trans, ASCE, Vol 113, PP, 425-444, 1948.

[10] **Timoshenko, S. and S. Woinowsky-Kreiger.** Theory of Plates and Shells, 2nd ed. New York : Mc Graw-Hill, 1959.

[11] **Horonjeff, R.** The planning and Design of Airports. Mc Graw-hill 1965.

[12] **Yoder, E.J. and M.W. Witczak,** Principles of Pavement Design, Wiley-New York, 1975.

[13] ICAO Aerodrome Design Manual, Part 3, Pavements, 1983.